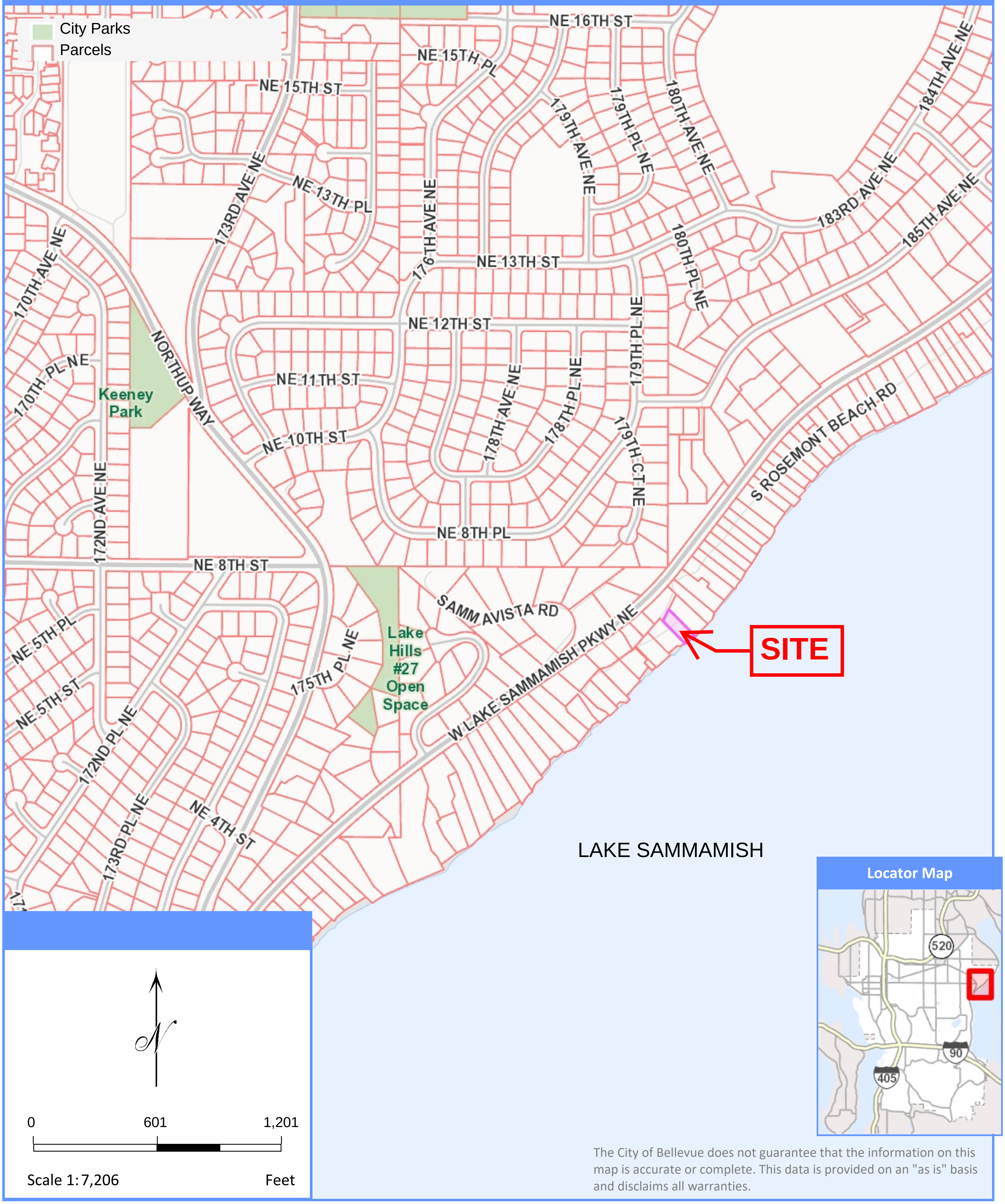
Vicinity Map



The City of Bellevue does not guarantee that the information on this map is accurate or complete. This data is provided on an "as is" basis and disclaims all warranties.

Map Generated on: 10/01/2020

NASH & ASSOCIATES
ARCHITECTS

Gary Nash
Michael Johnson
Marie Anderson

August 3, 2020

Project: Residence for Tom and Maggie Chan
Address: 672 W. Lk. Sammamish Pkwy. SE
Bellevue, WA 98008
Parcel No.: 312506-9008

Of Note: Under a previous owner this lot had gone through the Critical Areas Land Use Review process (Permit Number 16-129442-LO) and was given approval.

Project Narrative:

This is a multi-phase project being to be built in the City of Bellevue on the shoreline of Lake Sammamish.

Phase 1: The vacation of an existing 16'-0" access easement (Recording #AF: 4974043) and replacement with new 16'-0" access easement. This easement affects approximately 3 additional lots, as well as the subject property.

Phase 2: To run concurrently with Phase 1 will be the removal and replacement of an existing rockery on the property. The rockery is to be relocated and replaced by an engineered soldier pile wall.

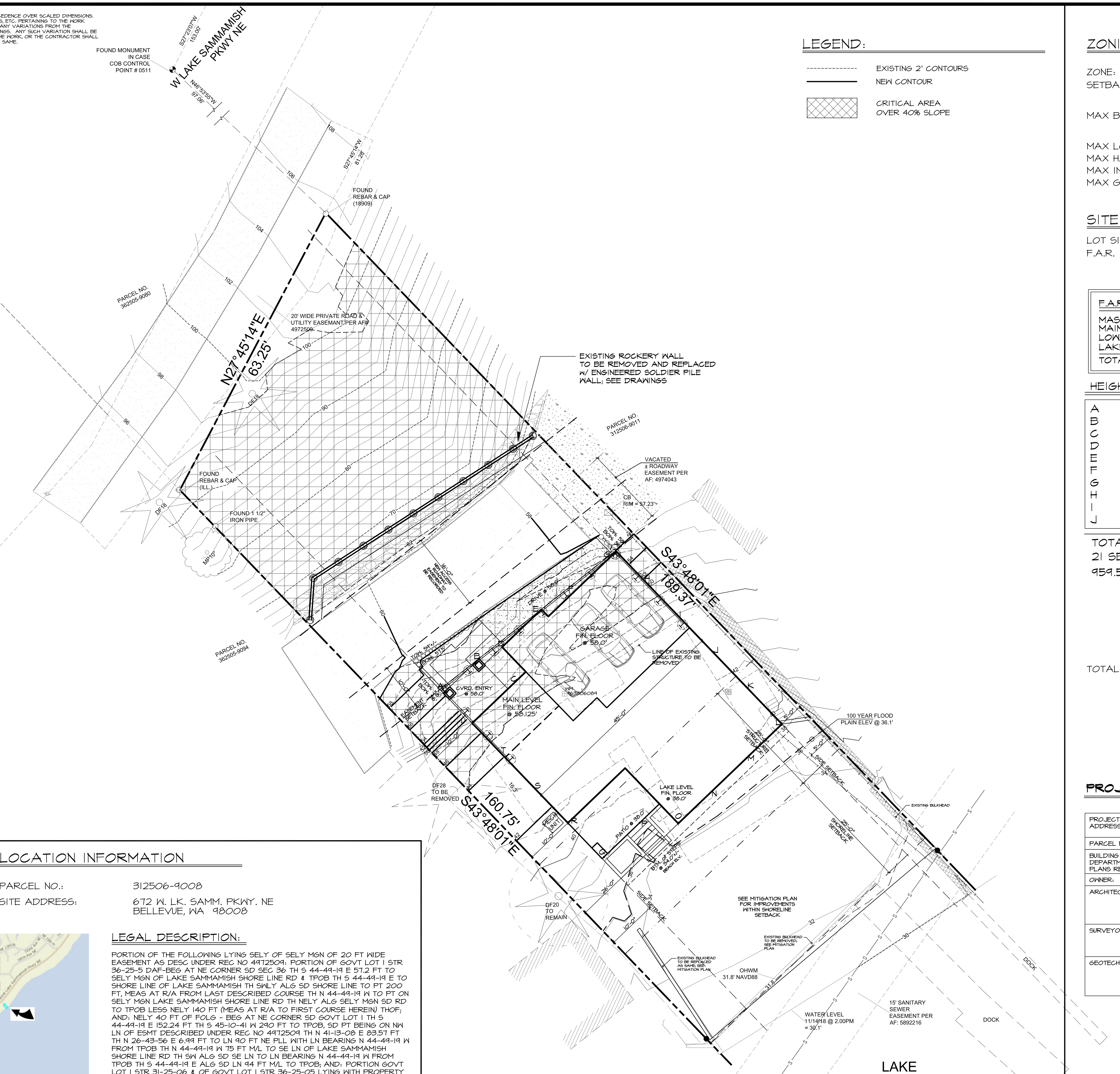
Phase 3: The existing 1958 structure that is in satisfactory condition is to be demolished and preliminary site work done to prepare for the construction of a new single-family residence. During this phase the shoreline mitigation work will be completed per the Shoreline and Critical Areas Mitigation Report prepared by Altman Oliver Associates.

Phase 4: The construction of a new 4,900 square foot 3 car garage, 5 bedroom single family residence. The design is to maximize the use and views of the site, while taking into account the difficult nature of siting a home on a narrow, steep lot located on Lake Sammamish.

Of Note: This lot had gone through a previous Critical Areas Land Use

Regards –
Gary Nash
Karen Mangold

WRITTEN DIMENSIONS ON THIS DRAWING SHALL HAVE PRECEDENCE OVER SCALED DIMENSIONS. CONTRACTOR SHALL VERIFY ALL DIMENSIONS, CONDITIONS, ETC. PERTAINING TO THE WORK BEFORE PROCEEDING. THE OWNER MUST BE NOTIFIED OF ANY VARIATIONS FROM THE DIMENSIONS AND/OR CONDITIONS SHOWN ON THESE DRAWINGS. ANY SUCH VARIATION SHALL BE RESOLVED BY THE OWNER PRIOR TO PROCEEDING WITH THE WORK, OR THE CONTRACTOR SHALL ACCEPT FULL RESPONSIBILITY FOR THE COST TO RECTIFY SAME.



OHWM for Lake Sammamish = 31.8'

LEGEND:

- EXISTING 2' CONTOURS
- NEW CONTOUR
- CRITICAL AREA OVER 40% SLOPE

ZONING DATA:

ZONE:	R - 2.5
SETBACKS:	FRONT - 20' REAR - 20' SIDE - TOTAL 15'
MAX BUILDING HEIGHT:	FLAT ROOF - 30' SLOPE ROOF - 35'
MAX LOT COVERAGE:	35%
MAX HARD SURFACE:	75%
MAX IMPERVIOUS:	45%
MAX GREEN SPACE:	50%

SITE DATA:

LOT SIZE:	10,413 SQ. FEET
F.A.R. = 50%	5,206 SQ. FT. MAX ALLOWABLE INCLUDING GARAGE'

F.A.R. SQUARE FOOTAGES:	
MASTER LEVEL	848
MAIN LEVEL	1,981
LOWER LEVEL	1,250
LAKE LEVEL	783
TOTAL	4,862

HEIGHT CALCULATION:

A	54'	K	41.5'	U	49'
B	55'	L	39'		
C	49.5'	M	39'		
D	54	N	39'		
E	54'	O	38'		
F	53'	P	37.5'		
G	54'	Q	38'		
H	51'	R	41'		
I	44'	S	42.5'		
J	43'	T	43.5'		

TOTAL 959.5
21 SEGMENTS
959.5/21 = 45.7' AVG. EXISTING GRADE

MAXIMUM HEIGHT ALLOWABLE:
FLAT = 75.7'
SLOPE = 80.7'

TOTAL IMPERVIOUS AREA: 3,670 sf (35%)

PROJECT INFORMATION

PROJECT ADDRESS:	224 - 9th Ave KIRKLAND, WA 98033
PARCEL NUMBER:	388580-4950
BUILDING DEPARTMENT & PLANS REVIEWER	CITY OF BELLEVUE
OWNER:	TOM & MAGGIE CHAN
ARCHITECT:	NASH & ASSOCAITES 11644 NE 80TH ST. KIRKLAND, WA 98033 TEL:425-828-4117 FAX:425-822-1918 GARY NASH & KAREN MANGOLD
SURVEYOR:	SITE SURVEYING, INC 21923 NE 11th ST. SAMMAMISH, WA 98074 TEL: 425-298-4412
GEOTECH:	GEOTECH CONSULTANTS, INC. 2401 10TH AVE E SEATTLE, WA 98102 TEL: 425-747-5618

SITE PLAN

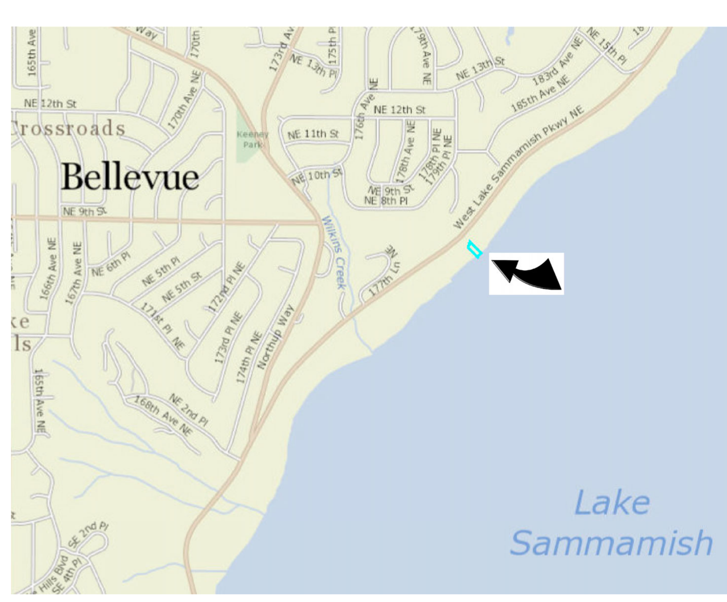
SEE GENERAL NOTES SCALE: 1" = 10'-0"

0 1 2 5 10 15 25 50

LOCATION INFORMATION

PARCEL NO.: 312506-9008

SITE ADDRESS: 672 W. LK. SAMM. PKWY. NE
BELLEVUE, WA 98008



VICINITY MAP
NOT TO SCALE

LEGAL DESCRIPTION:

PORTION OF THE FOLLOWING LYING SELY OF SELY MGN OF 20 FT WIDE EASEMENT AS DESC UNDER REC NO 4972509: PORTION OF GOVT LOT 1 STR 36-25-5 DAF-BEG AT NE CORNER SD SEC 36 TH S 44-49-19 E 57.2 FT TO SELY MGN OF LAKE SAMMAMISH SHORE LINE RD & TPOB TH S 44-49-19 E TO SHORE LINE OF LAKE SAMMAMISH TH SWLY ALG SD SHORE LINE TO PT 200 FT, MEAS AT R/A FROM LAST DESCRIBED COURSE TH N 44-49-19 W TO PT ON SELY MGN LAKE SAMMAMISH SHORE LINE RD TH NELY ALG SELY MGN SD RD TO TPOB LESS NELY 140 FT (MEAS AT R/A TO FIRST COURSE HEREIN) THOF; AND: NELY 40 FT OF FOLG - BEG AT NE CORNER SD GOVT LOT 1 TH S 44-49-19 E 152.24 FT TH S 45-10-41 W 290 FT TO TPOB, SD PT BEING ON NW LN OF ESMT DESCRIBED UNDER REC NO 4972509 TH N 41-13-08 E 83.57 FT TH N 26-43-56 E 6.99 FT TO LN 90 FT NE PLL WITH LN BEARING N 44-49-19 W FROM TPOB TH N 44-49-19 W 75 FT M/L TO SE LN OF LAKE SAMMAMISH SHORE LINE RD TH SW ALG SD SE LN TO LN BEARING N 44-49-19 W FROM TPOB TH S 44-49-19 E ALG SD LN 94 FT M/L TO TPOB; AND: PORTION GOVT LOT 1 STR 31-25-06 & OF GOVT LOT 1 STR 36-25-05 LYING WITH PROPERTY AS DESC UNDER REC NO 8602050919 & LYING NWLY OF SELY MGN OF 20 FT WIDE ESMT AS DESCRIBED IN REC NO 4972509; TGW PORTION GOVT LOT 1 STR 31-25-06 DAF-BEG AT NW CORNER SD SEC 31 TH S 44-49-19 E 57.2 FT TO SELY MGN OF LAKE SAMMAMISH SHORE LINE RD & TPOB TH S 44-49-19 E TO SHORE LINE OF LAKE SAMMAMISH TH SWLY ALG SD SHORE LINE TO PT 200 FT, MEAS AT R/A FROM LAST DESC COURSE TH N 44-49-19 W TO PT ON SELY MGN LAKE SAMMAMISH SHORE LN RD TH NELY ALG SELY MGN TO TPOB EXCEPT NELY 140 FT (MEAS AT R/A TO FIRST COURSE HEREIN) THOF TGW 2ND CLASS SHORELANDS ADJ -AKA LOT 2 CITY OF BELLEVUE BOUNDARY LINE ADJUSTMENT NO BLA-96-9076 REC NO 9702129006

NASH & ASSOCIATES
ARCHITECTS
11644 NE 80th STREET • KIRKLAND, WA • 98033 • 425-828-4117
www.nash-architects.com

project: CHAN PROPERTY
SITE PLAN - B
672 W Lk Samm Pkwy NE
BELLEVUE, WA

date: 08-03-20
permit:
revisions:

drawn by: KLM
checked by: GN

SHEET
SITE PLAN
OF
B

Altmann Oliver Associates, LLC

AOA

PO Box 578 Carnation, WA 98014 Office (425) 333-4535 Fax (425) 333-4509

Environmental Planning & Landscape Architecture

June 19, 2020

AOA-5309

Karen Mangold
karen@nash-architects.com

SUBJECT: **Critical Areas Report - Habitat Assessment**
672 W. Lake Samm. Pkwy NE, Bellevue, WA
Steep Slope and Shoreline Setback Modification and Enhancement
City of Bellevue File 16-129442-LO

Dear Karen:

On October 31, 2016 I conducted an initial wetland and stream reconnaissance and habitat assessment on the subject property located along the shoreline of the west side of Lake Sammamish. Additional field investigations were conducted by AOA in the spring of 2020. The primary purpose of the site visits was to assess proposed modifications to the steep slope, steep slope buffer, and shoreline habitat as part of a proposed re-development of the property to replace an existing residence with a new single-family residence. See the geotechnical report for information pertaining to slope stability and geotechnical performance standards.

No wetlands or streams were identified on the site utilizing the methodology outlined in the May 2010 *Regional Supplement to the Corps of Engineers Wetland Delineation Manual: Western Mountains, Valleys, and Coast Region (Version 2.0).*

1.0 EXISTING CONDITIONS

The central portion of the site is currently developed with a small single-family residence. A gravel beach area extends from the house to the lake edge and a newly installed dock and small concrete bulkhead. Vegetation surrounding the house consisted primarily of planted ornamental shrubs, salal (*Gaultheria shallon*), bamboo, and scattered Douglas fir (*Pseudotsuga menziesii*) and western red cedar (*Thuja plicata*) trees.

A steep slope is located throughout the western portion of the property along both sides of the access drive. The area west of the driveway consisted of scattered trees and brush and included Douglas fir, western red cedar, madrone (*Arbutus menziesii*), hazelnut (*Corylus cornuta*), Oregongrape (*Mahonia* sp.), and patches of invasive Himalayan blackberry (*Rubus armeniacus*).

No habitat features such as snags or large downed logs were observed.

2.0 CRITICAL AREA IMPACTS

Lake Sammamish requires a 50-foot shoreline structure setback from the ordinary high water of the lake adjacent Shoreline Residential uses per LUC Chart 20.25E.065.C.2. Steep slopes require a standard 75-foot buffer from the toe of the slope and a 50-foot buffer from the top of the slope.

2.1 Shoreline Structure Setback

The proposed new residence would be constructed in the same general location as the existing residence and there would be no expansion of the structure footprint within the 50-foot shoreline structure setback. Work that would occur within the shoreline setback includes the removal of an existing bulkhead and replacing the bulkhead with boulders and designated soft shoreline enhancements consisting of small trees, shrubs, and groundcover plantings.

The City of Bellevue requires that pre- and post-construction shoreline land uses utilize the credits and debits found in LUC Chart 20.25E.065.F.8.d.

<u>EXISTING CONDITIONS:</u>
IMPERVIOUS (INCLUDES DECK): 828 SF X 0.0 = 0.0
BARE GROUND OR PERVIOUS: 1,614 SF X 0.15 = 242.10
NON-NATIVE VEGETATION 25-50' FROM OHWL: 517 SF X 0.25 = 129.25
NON-NATIVE VEGETATION 0-25' FROM OHWL: 56 SF X 0.3 = 16.80

TOTAL: 388.15

<u>PROPOSED CONDITIONS:</u>
IMPERVIOUS: 824 SF X 0.0 = 0.0
BARE GROUND OR PERVIOUS: 751 SF X 0.15 = 112.65
NON-NATIVE VEGETATION 25-50' FROM OHWL: 283 SF X 0.25 = 70.75
NATIVE VEGETATION 25-50' FROM OHWL: 540 SF X 0.6 = 324.0
NATIVE VEGETATION 0-25' FROM OHWL: 617 SF X 0.80 = 493.60

TOTAL: 1,001.00

PROPOSED > EXISTING
1,001.0 > 388.15

2.2 Steep Slope and Steep Slope Buffer
The existing residence is located at the toe of the steep slope on the site. Because of topographic and shoreline buffer constraints, expansion of the residence is not possible without encroaching into this steep slope and its buffer. See the geotech report for an evaluation of the stability of the slope.

As part of the proposed project, expansion of the existing residence would encroach into 637 s.f. of steep slope and 260 s.f. of steep slope buffer adjacent the existing residence and driveway. The slope area that would be impacted consists primarily of yard that includes salal and ornamental plantings.

Development within a critical area steep slope and its buffer are subject to the applicable performance standards outlined in LUC 20.25H.125 (see geotech report). As part of these performance standards, all areas of disturbance within the critical area and its buffer must be mitigated per an approved mitigation/restoration plan.

3.0 CRITICAL AREA MITIGATION

3.1 Shoreline Structure Setback
Mitigation for work within the shoreline structure setback will occur through the removal of the existing bulkhead and the removal of designated portions of the gravel beach. Restoration would then include planting a variety of small trees, shrubs, and groundcover species within the degraded shoreline to increase the habitat value of the shoreline environment.

Planting the area with native species would increase the plant species and structural diversity over existing conditions and will increase food chain support by increasing the biological production of both vegetation and insects. This in turn should provide food and cover for a variety of song birds and other wildlife and increase the habitat function of the shoreline.

As part of the plan, native vegetation would be planted adjacent the shoreline. This would provide some limited natural shade to the lake. These plantings would also create some overhead cover while contributing detritus and other desirable allochthonous inputs into the aquatic environment.

3.2 Steep Slope and Buffer
As part of the proposed project, selected existing steep slope and buffer areas throughout the western portion of the site would be restored and enhanced by: 1) removing Himalayan blackberry and other invasive species, and 2) planting with a variety of native species to increase the plant species and structural diversity of the slope. This planting will increase the overall habitat value of the slope over current conditions.

3.3 Goal, Objectives, and Performance Standards for Mitigation Areas
The primary goal of the mitigation plan is to increase the habitat functions of the selected shoreline buffer and slope areas. To meet this goal, the following objectives and performance standards have been incorporated into the design of the plan:

Objective A: Increase the structural and plant species diversity within the mitigation area.
Performance Standard: There will be 100% survival of all woody planted species throughout the mitigation area at the end of the first year of planting. For Years 2-5, success will be based on an 85% survival rate or similar number of recolonized native woody plants. Areal coverage of plantings or native re-colonized species will be at least 15% at Year 1, 20% at Year 2, 25% at Year 3, and 40% at Year 5.

Objective B: Limit the amount of invasive and exotic species within the mitigation area.
Performance Standard: After construction and following every monitoring event for a period of five years, exotic and invasive plant species will be maintained at levels below 10% total cover in the designated mitigation areas. Invasive species include, but are not limited to, Himalayan and evergreen blackberry, Japanese knotweed, and English ivy.

3.4 Construction Management
Prior to commencement of any work in the mitigation areas, the clearing limits will be staked and any existing vegetation to be saved will be clearly marked. A pre-construction meeting will be held at the site to review and discuss all aspects of the project with the landscape contractor and the owner.

A consultant will supervise plan implementation during construction to ensure that objectives and specifications of the mitigation plan are met. Any necessary significant modifications to the design that occur as a result of unforeseen site conditions will be jointly approved by the City of Bellevue and the consultant prior to their implementation.

3.5 Monitoring Methodology
The monitoring program will be conducted for a period of five years, with annual reports submitted to the City. Vegetation monitoring will include general appearance, health, mortality, colonization rates, percent cover, percent survival, volunteer plant species, and invasive weeds.

Photo-points will be established from which photographs will be taken throughout the monitoring period. These photographs will document general appearance and progress in plant community establishment in the mitigation area. Review of the photos over time will provide a visual representation of success of the mitigation plan.

3.6 Maintenance Plan
Maintenance will be conducted on a routine, year-round basis. Additional maintenance needs will be identified and addressed following periodic maintenance reviews. Contingency measures and remedial action on the site shall be implemented on an as-needed basis at the direction of the consultant or the owner.

3.7 Weed Control
Routine removal and control of non-native and other invasive plants within the designated mitigation areas shall be performed by manual means. Undesirable and weedy exotic plant species shall be maintained at levels below 10% total cover within all mitigation areas during the monitoring period.

3.8 General Maintenance Items
Routine maintenance of planted trees and shrubs shall be performed. Measures include resetting plants to proper grades and upright positions. Tall grasses and other competitive weeds shall be weeded at the base of plants to prevent engulfment. Weed control should be performed by hand removal.

3.9 Contingency Plan
All dead plants will be replaced with the same species or an approved substitute species that meets the goal of the mitigation plan. Plant material shall meet the same specifications as originally installed material. Replanting will not occur until after reason for failure has been identified (e.g., moisture regime, poor plant stock, disease, shade/sun conditions, wildlife damage, etc.). Replanting shall be completed under the direction of the consultant, City of Bellevue, or the owner.

3.10 As-Built Plan
Following completion of construction activities, an as-built plan for the mitigation area will be provided to the City of Bellevue. The plan will identify and describe any changes in relation to the original approved plan.

4.0 FUNCTIONAL ASSESSMENT TOOL
The project site was evaluated using the City of Bellevue's *Draft Functional Assessment Tool for Upland Habitat* (**Attachment A**). Based on this assessment the project site received a score of 29. In general, sites with scores between 26 and 40 "provide both actual habitat and likely the opportunity for wildlife to use the habitat on the site."

The project site received relatively high scores for its proximity to Lake Sammamish and by the presence of large coniferous trees.

Limiting factors on the site included the lack of habitat features and a relatively low vegetative vertical structural diversity. In addition, the site is entirely surrounded by development and effectively disconnected from other habitats. Furthermore, the existing shoreline of Lake Sammamish is developed and does not provide a significant habitat area.

Since no significant native plant communities would be removed as part of the proposed project, there are no anticipated impacts to any species of local importance.

If you have any questions regarding the critical areas study, please give me a call.

Sincerely,

ALTMANN OLIVER ASSOCIATES, LLC

John Altmann

John Altmann
Ecologist

ATTACHMENT A
DRAFT FUNCTIONAL ASSESSMENT TOOL FOR UPLAND HABITAT

City of Bellevue

DRAFT FUNCTIONAL ASSESSMENT TOOL

for Upland Habitat

Property address 672 W. LAKE SAMM. PKWY NE

Location Range 6E Township 25N Section 31

Parcel number 312506-9008

Property owner VILMA STOSS

Telephone number ( ) - -

Project name

Project contact JOHN ALTMANN

Telephone number (425) -333 -4535

Address PO BOX 578 CARNATION, WA 98014

Staff JOHN ALTMANN

Date(s) of site visit(s) 10/31/16

Washington Department of Fish and Wildlife Priority Habitat and Species (PHS) data obtained? Y/N YES

1.0	PROPERTY DESIGNATION	Zone A	Zone B	Zone C	Zone D		Zone
1.1	Existing impervious surface	>90%	50-90%	20-50%	0-20%		C
2.0	LANDSCAPE PARAMETERS	No points	1 point	2 points	3 points	Additional points	Total
2.1	Land use/development density	Zone A	Zone B	Zone C	Zone D		2
2.2	*Occurrence (number) of habitat types	0	1	2	3+		2
2.3	**Proximity of known critical areas (distance to edge)	>2,500 ft	<2,500 ft	<1,200 ft	<100 ft	+1 point if contiguous with critical area	4
2.4	Habitat connectivity and corridors	No connection to other habitat areas	≥50-foot-wide connection to vegetated areas of at least 1 acre	≥50-foot-wide connection to vegetated areas of at least 50 acres but not listed parks***	≥50-foot-wide connection King County wildlife network or listed parks***	+1 point for ≥150-foot-wide connection King County wildlife network or listed parks***	0
2.5	Patch size	<0.-1.0 ac	1.0-5.0 ac	>5-10 ac	10-42 acres	>42 acres = 4 points	1

City of Bellevue

DRAFT FUNCTIONAL ASSESSMENT TOOL

for upland habitat

2.0	LANDSCAPE PARAMETERS	No points	1 point	2 points	3 points	Additional points	Total
2.6	***Interspersion of habitat patches (excluding patches <1 ac in area)**	No or isolated patch (no others within 0.5-ac circle)	Low	Moderate	High	+1 point if wildlife network or listed park is included	1
3.0	**LOCAL PARAMETERS**	**No points**	**1 point**	**2 points**	**3 points**	**Additional points**	**Total**
3.1	**Size of native trees on site**	No significant trees on site	6-12" dbh tree(s) present	12-20" dbh tree(s) present	>20" dbh tree(s) present	+1 point if tree(s) >30" dbh are present	3
3.2	**Coniferous component**	No conifers on site	Conifers very sparse or present in understory only	Conifers co- or sub-dominant in overstory only 4 trees	Conifers dominant	+1 point if conifers >30" dbh are present	2
3.3	**Percent cover (sample vegetated areas only)**						
	Ground layer (0-2.3 ft) (5-ft radius)	0%	0-25%	25-50%	50%+	+1 point for cover >75%; -1 point if mowed grass is >50%	3
	Shrub layer (2.3-25 ft) (10-ft radius)	0%	0-25%	25-50%	50%+	+1 point for cover >75%	2
	Canopy (>25 ft) (30-ft radius)	0%	0-25%	25-50%	50%+	+1 point for cover >75%	2
3.4	**Vegetative vertical structural diversity (foliage height diversity)**	FHD = 0	FHD < 0.70	FHD = 0.70-0.90	FHD > 0.90		1
3.5	**Vegetative species richness**	0-1 species	2-5 species	6-19 species	20+ species		2
3.6	**Invasive species component**	>75% cover	25-75% cover	10-25%cover	<10% cover		1

City of Bellevue

DRAFT FUNCTIONAL ASSESSMENT TOOL

for Upland Habitat

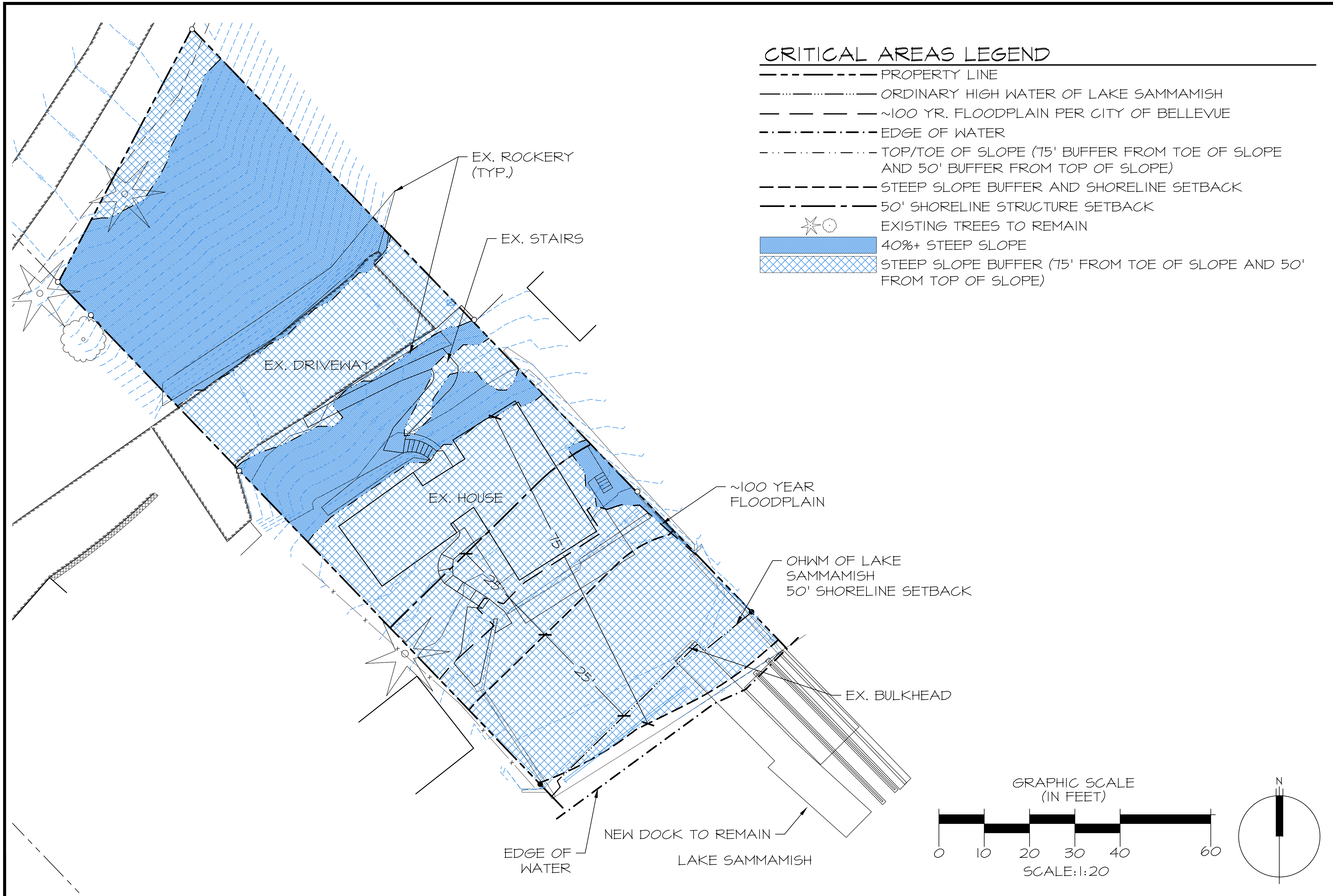
3.0	LOCAL PARAMETERS	No points	1 point	2 points	3 points	Additional points	Total
3.7	**Proximity to year-round water**	>1.0 mi or artificial feature with maintained /invasive buffer present within 0.3-1 mi	0.3-1.0 mi or artificial feature with maintained/ invasive buffer present within <0.3 mi	<0.3 mi or artificial feature with maintained/ invasive buffer present within patch	Natural water feature present within patch with native buffer		2
3.8	**Snags (≥4 in dbh)**	No snags on site	1/ac or fewer	2-6/ac	>7/ac	Add 0.5 point for each >20 in dbh and 1 point for each >30 in dbh	0
3.9	**Other habitat features**	None	1	2-4	5 or more		1
Landscape parameters points							10
Local parameters points							18
TOTAL POINTS							28

* Use circle of the appropriate size for the property's zone:

Zone A – 0.5 ac
Zone B – 5.0 ac
Zone C – 100 ac
Zone D – 250 ac

** PHS data required for sites in Zone D

***Parks: Mercer Slough, Phantom Lake wetland complex, Larson Lake wetland complex, Cougar Mountain Regional Wildland Park, Weowna Park; King County wildlife network



PROJECT 5309

DRAWN SO

SCALE AS NOTED

DATE 5-28-20

REVISED 6-08-20

1/11

FIGURE 1: EXISTING CONDITIONS MAP
BUFFER MITIGATION PLAN
672 W. LAKE SAMMAMISH PKWY. NE
BELLEVUE, WASHINGTON

AOA Environmental Planning & Landscape Architecture
Altmann Oliver Associates, LLC
PO Box 578 Carnation, WA 98014 Office (425) 333-4535 Fax (425) 333-4509

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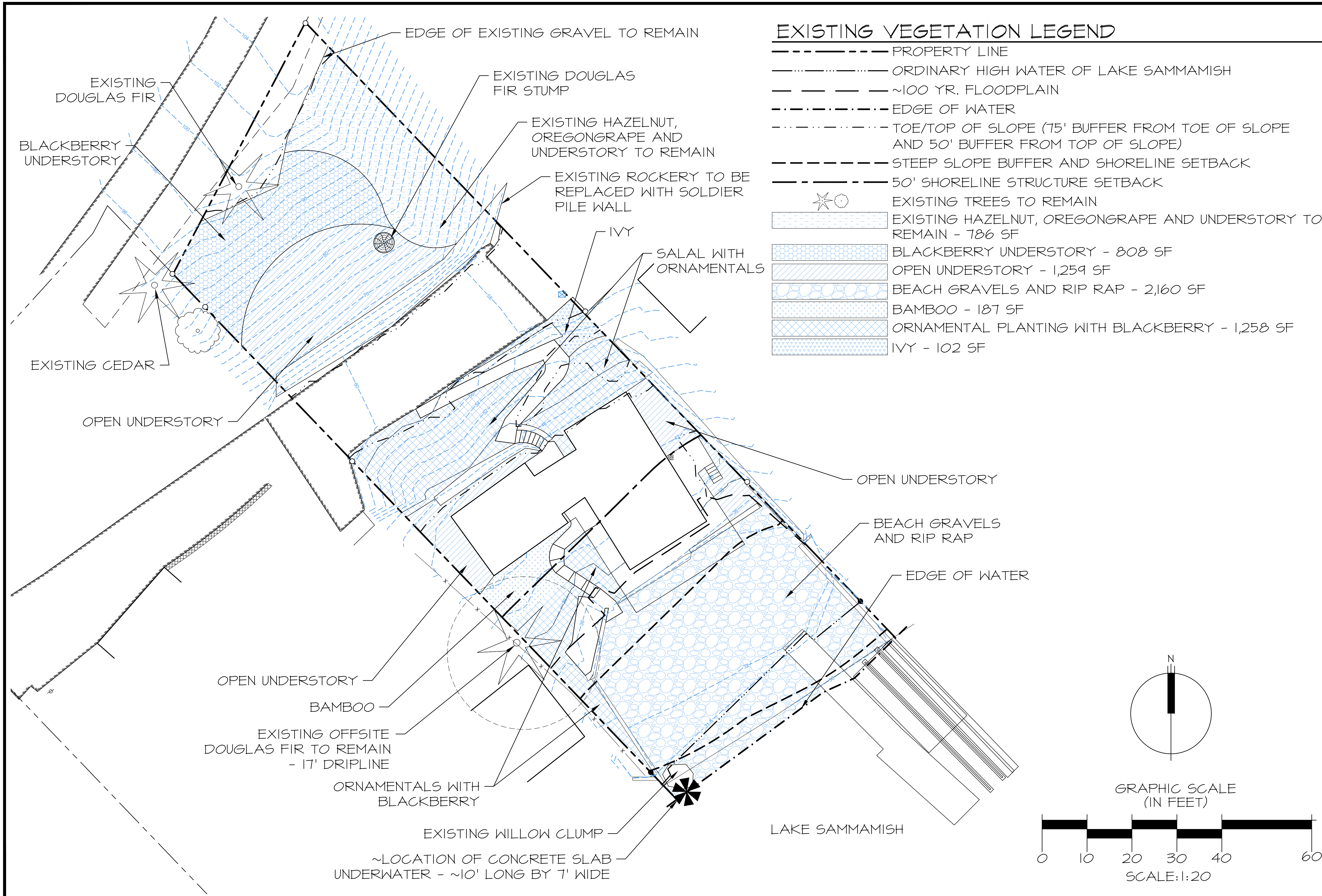

PROJECT 5309
DRAWN SO
SCALE AS NOTED
DATE 5-28-20
REVISED 6-08-20
2/11

FIGURE 2: EXISTING VEGETATION MAP
BUFFER MITIGATION PLAN
672 W. LAKE SAMMAMISH PKWY. NE
BELLEVUE, WASHINGTON

AOA Environmental Planning & Landscape Architecture
Altmann Oliver Associates, LLC
PO Box 578 Carnation, WA 98014 Office (425) 333-4535 Fax (425) 333-4509

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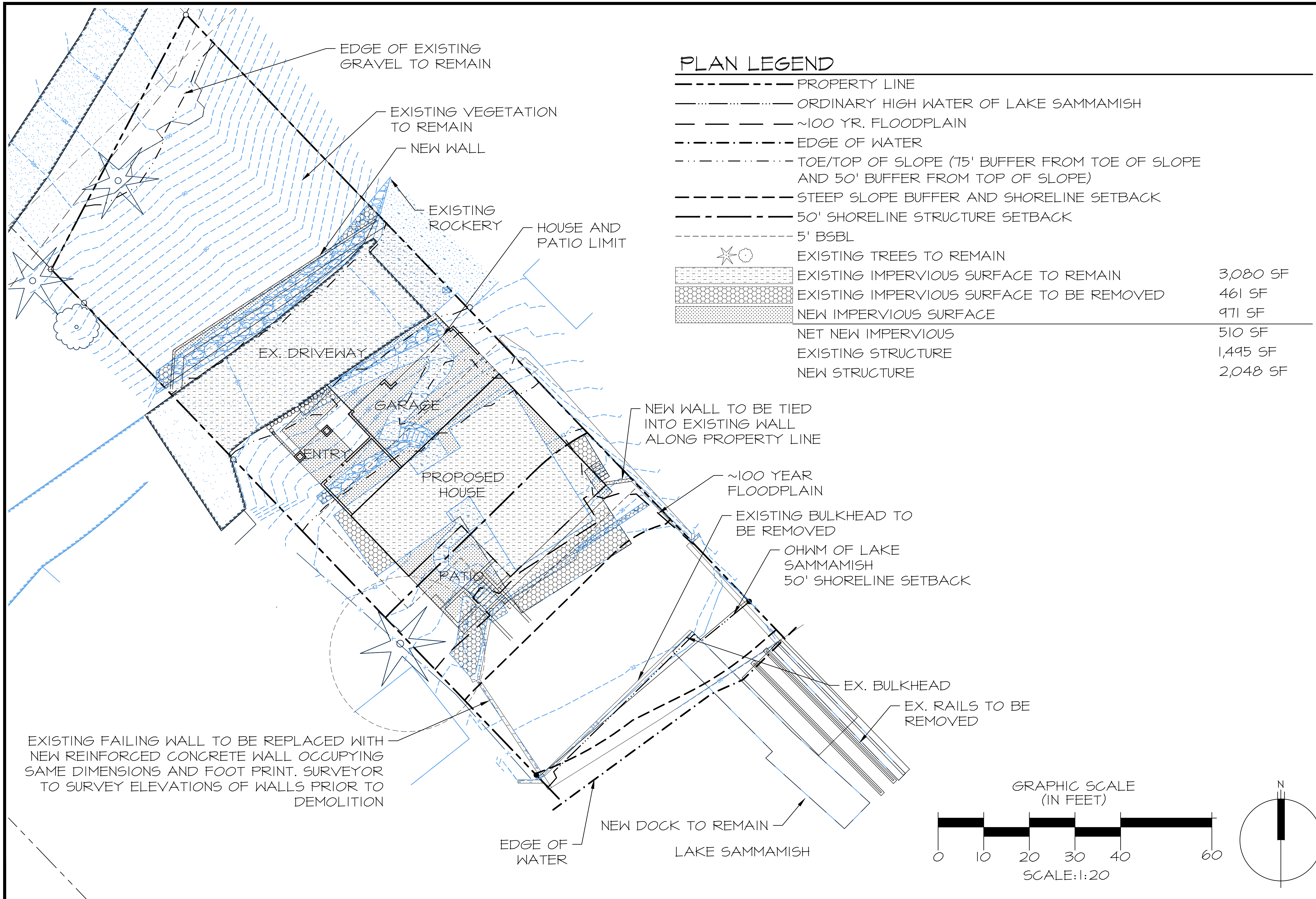

PLAN LEGEND

(line)	PROPERTY LINE	
(line)	ORDINARY HIGH WATER OF LAKE SAMMAMISH	
(line)	~100 YR. FLOODPLAIN	
(line)	EDGE OF WATER	
(line)	TOE/TOP OF SLOPE (75' BUFFER FROM TOE OF SLOPE AND 50' BUFFER FROM TOP OF SLOPE)	
(line)	STEEP SLOPE BUFFER AND SHORELINE SETBACK	
(line)	50' SHORELINE STRUCTURE SETBACK	
(line)	5' BSBL	
(symbol)	EXISTING TREES TO REMAIN	
(hatch)	EXISTING IMPERVIOUS SURFACE TO REMAIN	3,080 SF
(hatch)	EXISTING IMPERVIOUS SURFACE TO BE REMOVED	461 SF
(hatch)	NEW IMPERVIOUS SURFACE	971 SF
	NET NEW IMPERVIOUS	510 SF
	EXISTING STRUCTURE	1,495 SF
	NEW STRUCTURE	2,048 SF

PROJECT 5309
DRAWN SO
SCALE AS NOTED
DATE 5-28-20
REVISED 6-08-20
3/11

FIGURE 3: PROPOSED SITE PLAN
BUFFER MITIGATION PLAN
672 W. LAKE SAMMAMISH PKWY. NE
BELLEVUE, WASHINGTON

AOA Environmental Planning & Landscape Architecture
Altmann Oliver Associates, LLC
PO Box 578 Carnation, WA 98014 Office (425) 333-4535 Fax (425) 333-4509

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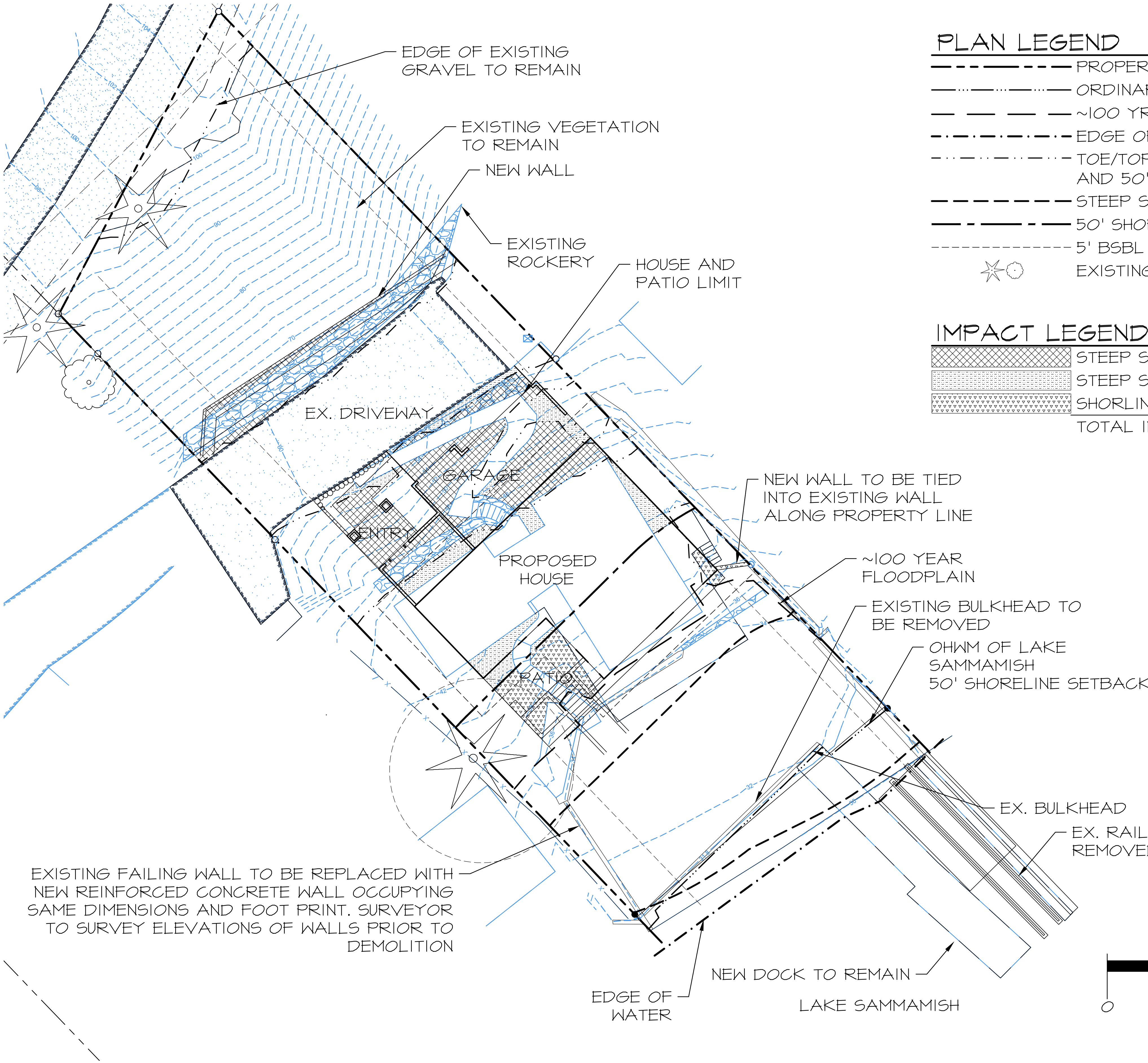
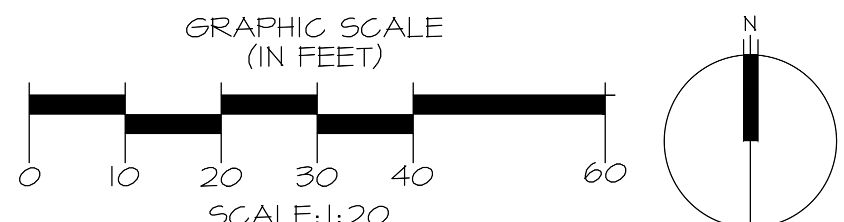


PLAN LEGEND

- PROPERTY LINE
- ORDINARY HIGH WATER OF LAKE SAMMAMISH
- ~100 YR. FLOODPLAIN
- EDGE OF WATER
- TOE/TOP OF SLOPE (75' BUFFER FROM TOE OF SLOPE AND 50' BUFFER FROM TOP OF SLOPE)
- STEEP SLOPE BUFFER AND SHORELINE SETBACK
- 50' SHORELINE STRUCTURE SETBACK
- 5' BSBL
- EXISTING TREES TO REMAIN

IMPACT LEGEND

STEEP SLOPE IMPACTS	637 SF
STEEP SLOPE BUFFER IMPACTS	260 SF
SHORLINE SETBACK IMPACTS	161
TOTAL IMPACTS	1,058 SF



PROJECT 5309
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FIGURE 4: CRITICAL AREAS IMPACTS
BUFFER MITIGATION PLAN
672 W. LAKE SAMMAMISH PKWY. NE
BELLEVUE, WASHINGTON

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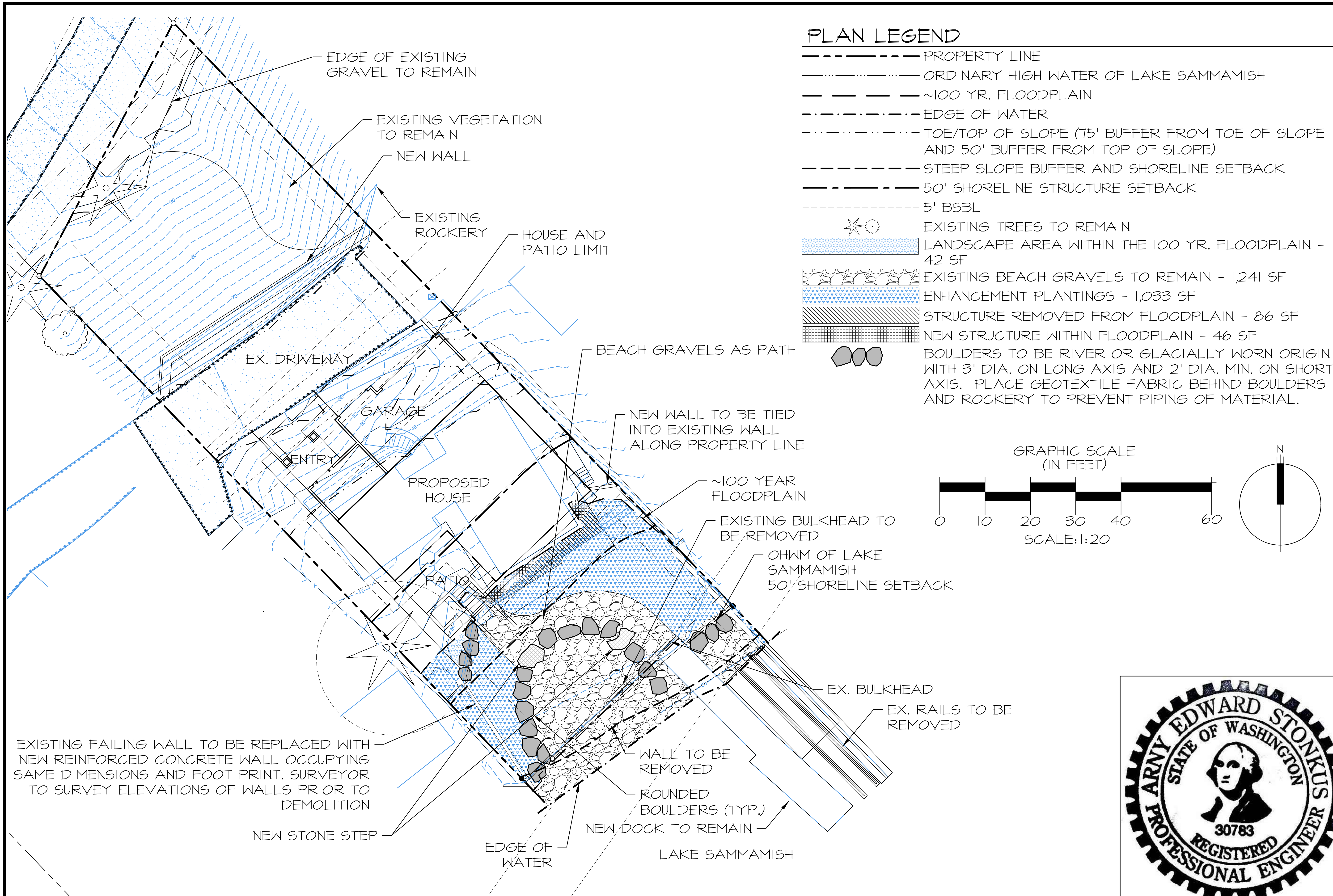

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FIGURE 5: WORK WITHIN THE 100-YEAR FLOOD PLAIN
BUFFER MITIGATION PLAN
672 W. LAKE SAMMAMISH PKWY. NE
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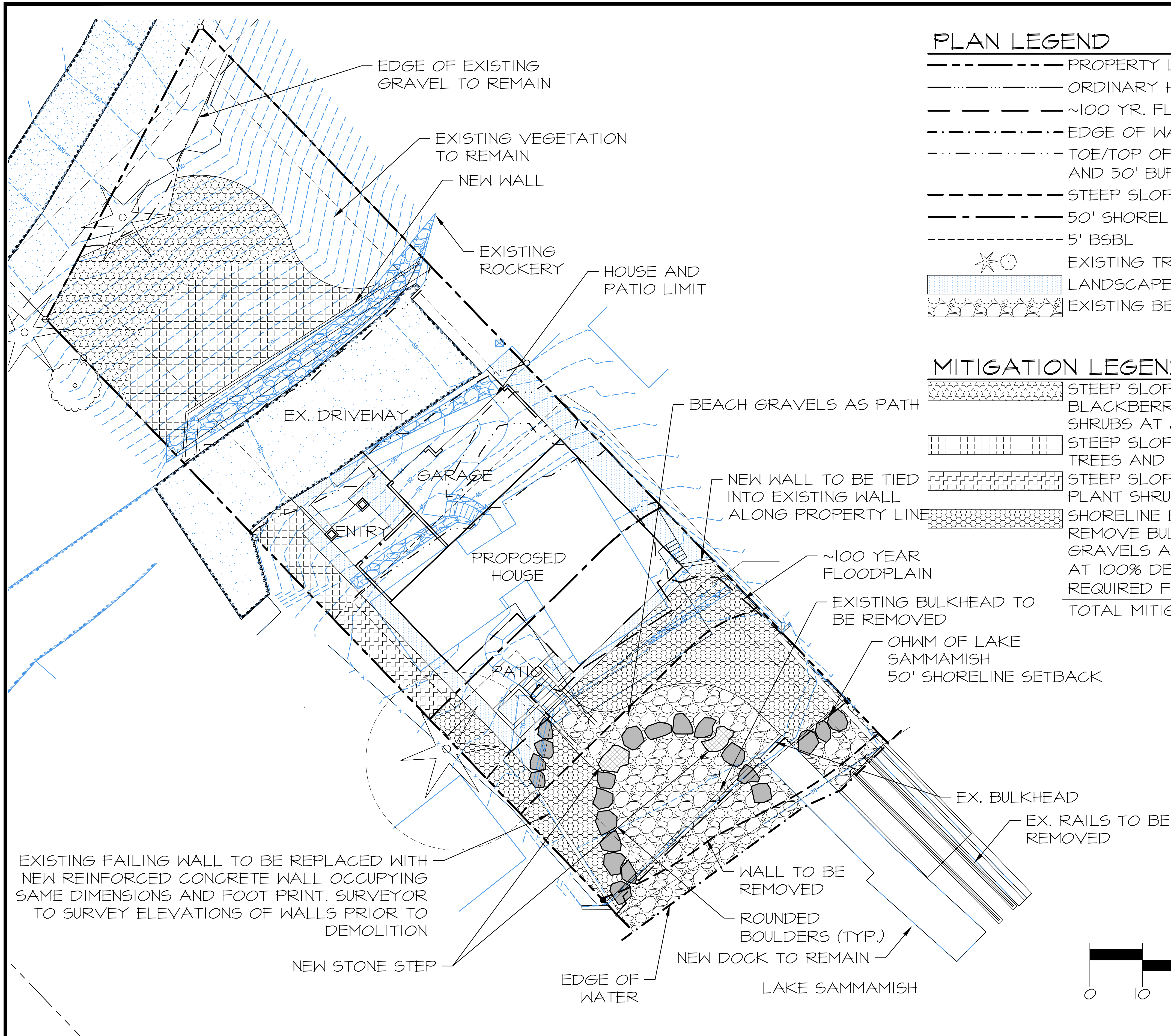
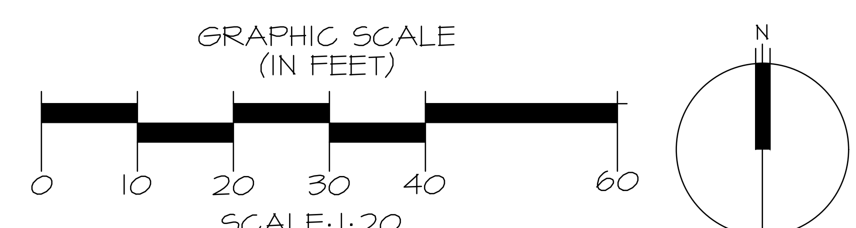


PLAN LEGEND

- PROPERTY LINE
- ORDINARY HIGH WATER OF LAKE SAMMAMISH
- ~100 YR. FLOODPLAIN
- EDGE OF WATER
- TOE/TOP OF SLOPE (75' BUFFER FROM TOE OF SLOPE AND 50' BUFFER FROM TOP OF SLOPE)
- STEEP SLOPE BUFFER AND SHORELINE SETBACK
- 50' SHORELINE STRUCTURE SETBACK
- 5' BSBL
- EXISTING TREES TO REMAIN
- LANDSCAPE AREA - 742 SF
- EXISTING BEACH GRAVELS TO REMAIN - 1,241 SF

MITIGATION LEGEND

STEEP SLOPE ENHANCEMENT - REMOVE BLACKBERRY AND PLANT TREES AND SHRUBS AT 80% DENSITY	808 SF
STEEP SLOPE ENHANCEMENT - PLANT TREES AND SHRUBS AT 100% DENSITY	945 SF
STEEP SLOPE BUFFER ENHANCEMENT - PLANT SHRUBS AT 100% DENSITY	199 SF
SHORELINE BUFFER RESTORATION - REMOVE BULKHEAD, WALLS AND BEACH GRAVELS AND PLANT TREES AND SHRUBS AT 100% DENSITY (INCLUDES PLANTING REQUIRED FOR DOCK PERMIT)	1,164 SF
TOTAL MITIGATION	3,116 SF



SCALE: 1:20

FIGURE 6: CRITICAL AREAS MITIGATION PLAN
BUFFER MITIGATION PLAN
672 W. LAKE SAMMAMISH PKWY. NE
BELLEVUE, WASHINGTON

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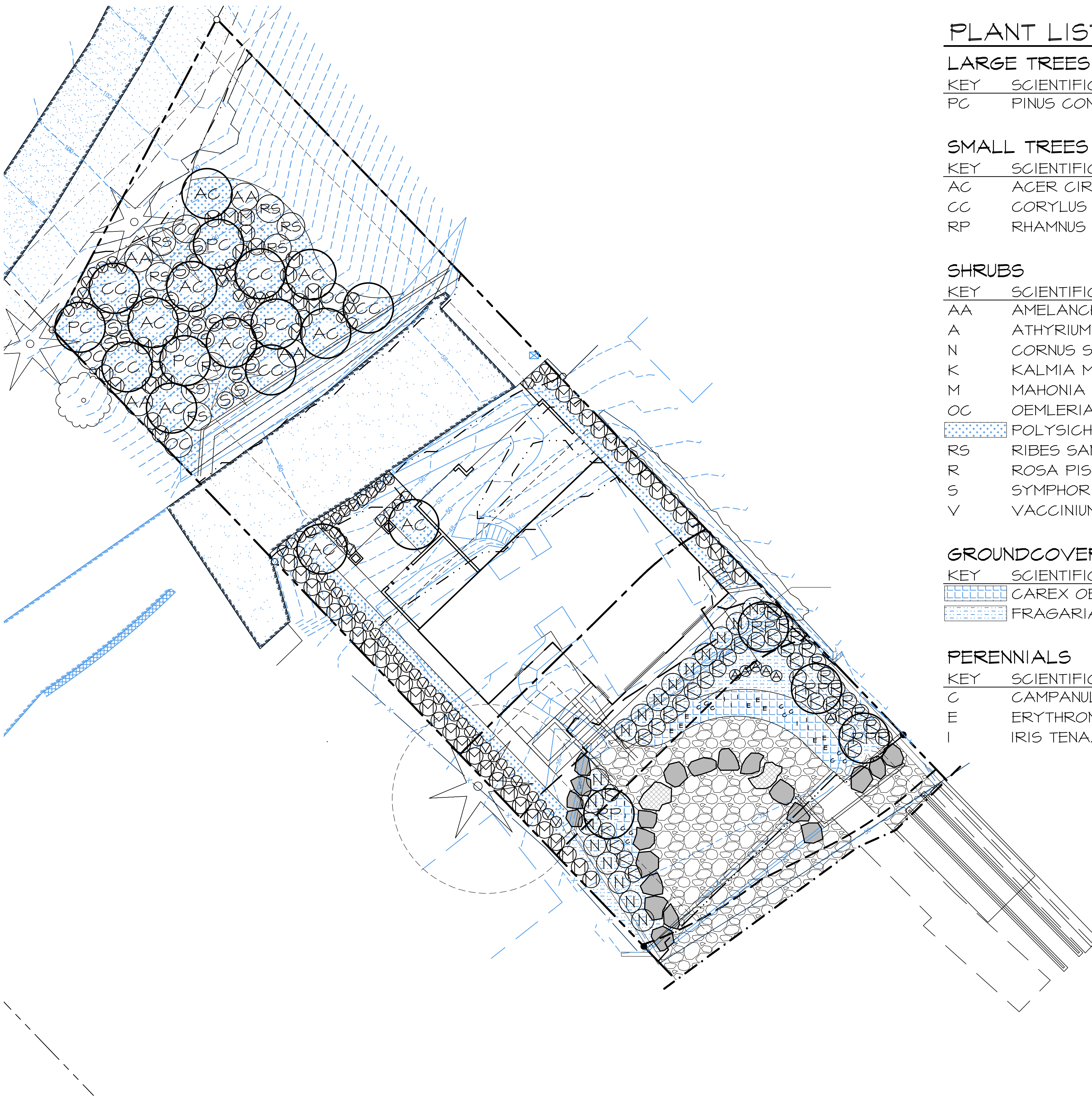


PLANT LIST (SEE FIGURE 8 FOR COMPLETE SCHEDULE)

LARGE TREES

KEY	SCIENTIFIC NAME	COMMON NAME
PC	PINUS CONTORTA	SHORE PINE

SMALL TREES

KEY	SCIENTIFIC NAME	COMMON NAME
AC	ACER CIRCINATUM	VINE MAPLE
CC	CORYLUS CORNUTA	WESTERN HAZELNUT
RP	RHAMNUS PURSHIANA	CASCARA

SHRUBS

KEY	SCIENTIFIC NAME	COMMON NAME
AA	AMELANCHIER ALNIFOLIA	SERVICEBERRY
A	ATHYRIUM FILIX-FEMINA	LADY FERN
N	CORNUS SERICEA	RED-OSIER DOGWOOD
K	KALMIA MICROPHYLLA	BOG LAUREL
M	MAHONIA AQUIFOLIUM	TALL OREGON GRAPE
OC	OEMLERIA CERASIFORMIS	INDIAN PLUM
	POLYSICHUM MUNITUM	SWORD FERN
RS	RIBES SANGUINEUM	RED CURRANT
R	ROSA PISOCARPA	CLUSTERED ROSE
S	SYMPHORICARPOS ALBUS	SNOWBERRY
V	VACCINIUM OVATUM	EVERGREEN HUCKLEBERRY

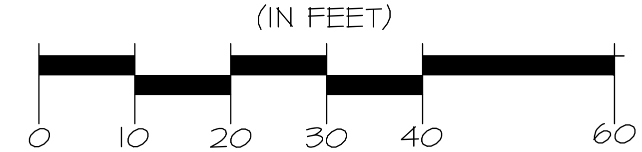
GROUNDCOVER

KEY	SCIENTIFIC NAME	COMMON NAME
	CAREX OBNUPTA	SLOUGH SEDGE
	FRAGARIA CHILOENSIS	COAST STRAWBERRY

PERENNIALS

KEY	SCIENTIFIC NAME	COMMON NAME
C	CAMPANULA ROTUNDIFOLIA	COMMON HAIRBELL
E	ERYTHRONIUM MONTANUM	AVALANCHE LILY
I	IRIS TENAX	OREGON IRIS

GRAPHIC SCALE
(IN FEET)



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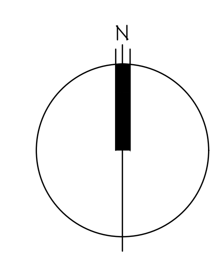

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FIGURE 7: STEEP SLOPE PLANTING PLAN
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PLANT SCHEDULE

LARGE TREES

KEY	SCIENTIFIC NAME	COMMON NAME	SPACING	QTY.	SIZE (MIN.)	NOTES
PC	PINUS CONTORTA	SHORE PINE	10' O.C.	4	2 GAL.	FULL & BUSHY

SMALL TREES

KEY	SCIENTIFIC NAME	COMMON NAME	SPACING	QTY.	SIZE (MIN.)	NOTES
AC	ACER CIRCINATUM	VINE MAPLE	10' O.C.	9	2 GAL.	MULTI-STEM (3 MIN.)
CC	CORYLUS CORNUTA	WESTERN HAZELNUT	10' O.C.	5	2 GAL.	MULTI-STEM (3 MIN.)
RP	RHAMNUS PURSHIANA	CASCARA	10' O.C.	4	2 GAL.	MULTI-STEM (3 MIN.)

SHRUBS

KEY	SCIENTIFIC NAME	COMMON NAME	SPACING	QTY.	SIZE (MIN.)	NOTES
AA	AMELANCHIER ALNIFOLIA	SERVICEBERRY	5' O.C.	4	1 GAL.	MULTI-STEM (3 MIN.)
A	ATHYRIUM FILIX-FEMINA	LADY FERN	2' O.C.	6	1 GAL.	FULL & BUSHY
N	CORNUS SERICEA	RED-OSIER DOGWOOD	4' O.C.	17	1 GAL.	MULTI-STEM (3 MIN.)
K	KALMIA MICROPHYLLA	BOG LAUREL	3' O.C.	25	1 GAL.	MULTI-STEM (3 MIN.)
M	MAHONIA AQUIFOLIUM	TALL OREGON GRAPE	3' O.C.	69	1 GAL.	FULL & BUSHY
OC	OEMLERIA CERASIFORMIS	INDIAN PLUM	5' O.C.	4	1 GAL.	MULTI-STEM (3 MIN.)
	POLYSICHUM MUNITUM	SWORD FERN	3' O.C.	114	1 GAL.	FULL & BUSHY
RS	RIBES SANGUINEUM	RED CURRANT	5' O.C.	8	1 GAL.	MULTI-STEM (3 MIN.)
R	ROSA PISOCARPA	CLUSTERED ROSE	3' O.C.	11	1 GAL.	MULTI-STEM (3 MIN.)
S	SYMPHORICARPOS ALBUS	SNOWBERRY	3' O.C.	24	1 GAL.	MULTI-STEM (3 MIN.)
V	VACCINIUM OVATUM	EVERGREEN HUCKLEBERRY	2' O.C.	79	1 GAL.	FULL & BUSHY

GROUNDCOVER

KEY	SCIENTIFIC NAME	COMMON NAME	SPACING	QTY.	SIZE (MIN.)	NOTES
	CAREX OBNUPTA	SLOUGH SEDGE	1.5' O.C.	57	CLUMP DIVISION	FULL & BUSHY
	FRAGARIA CHILOENSIS	COAST STRAWBERRY	2' O.C.	203	4" POT	FULL & BUSHY

PERENNIALS

KEY	SCIENTIFIC NAME	COMMON NAME	SPACING	QTY.	SIZE MIN.)	NOTES
C	CAMPANULA ROTUNDIFOLIA	COMMON HAIRBELL	1' O.C.	12	1 GAL. OR 4" POT	FULL & BUSHY
E	ERYTHRONIUM MONTANUM	AVALANCHE LILY	6" O.C.	9	1 GAL. OR 4" POT	FULL & BUSHY
I	IRIS TENAX	OREGON IRIS	1' O.C.	9	1 GAL. OR 4" POT	FULL & BUSHY

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FIGURE 8: STEEP SLOPE PLANT SCHEDULE
BUFFER MITIGATION PLAN
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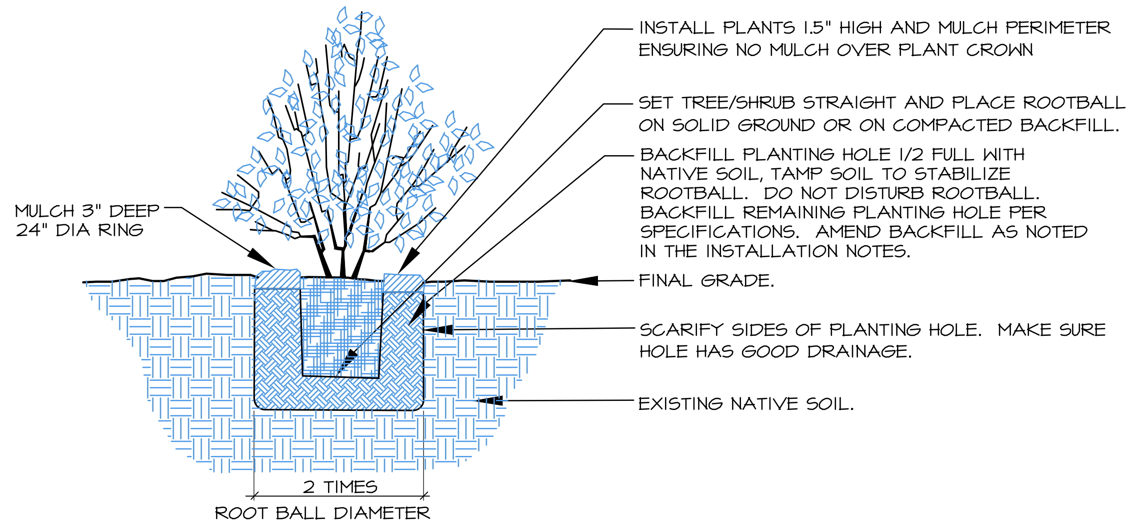
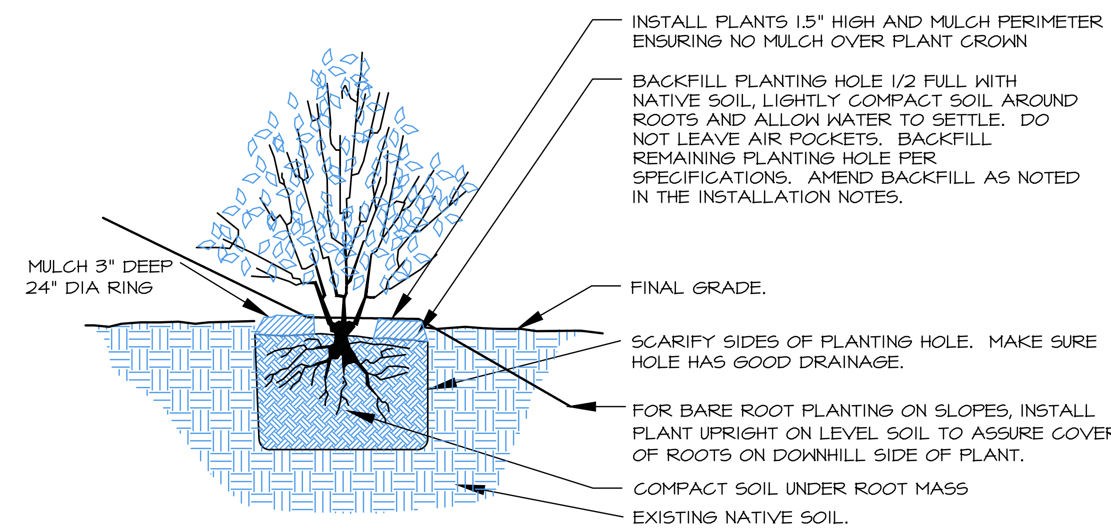


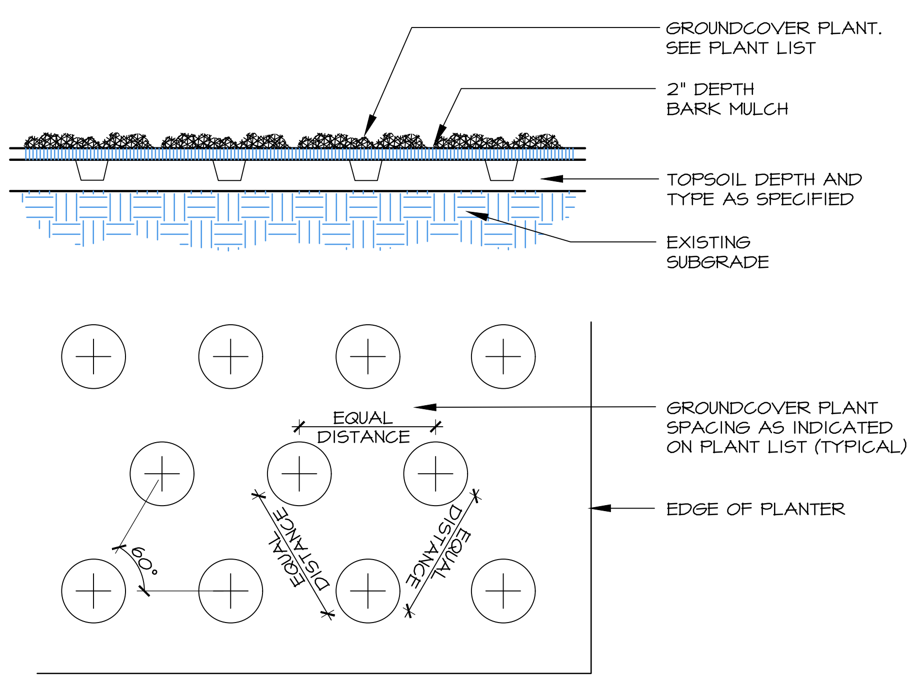
1 CONTAINER TREE/SHRUB PLANTING (TYP.)

SCALE: NTS



2 BARE-ROOT SHRUB PLANTING (TYP.)

SCALE: NTS



3 GROUNDCOVER PLANTING (TYP.)

SCALE: NTS

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FIGURE 9: PLANTING DETAILS
BUFFER MITIGATION PLAN
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SPECIFICATIONS

1. THIS PLAN PERTAINS TO PLANTING PORTION OF THE SITE WORK ONLY.
2. CONTRACTOR INFORMATION. WHEN IT IS AVAILABLE, CONTACT INFORMATION SHALL BE PROVIDED TO THE CITY OF BELLEVUE THAT INCLUDES NAMES, ADDRESSES AND PHONE NUMBERS OF PERSONS/FIRMS THAT WILL BE RESPONSIBLE FOR INSTALLING REQUIRED PLANTS AND PERFORMING REQUIRED MAINTENANCE.
3. CONTRACTOR'S QUALIFICATIONS. ALL WORK SHALL BE PERFORMED BY A LICENSED LANDSCAPE CONTRACTOR REGISTERED IN THE STATE OF WASHINGTON. CONTRACTOR MUST BE EXPERIENCED IN MITIGATION AND RESTORATION WORK. THE CONTRACTOR SHALL PROVIDE THAT THERE IS ONE PERSON ON THE SITE AT ALL TIMES DURING WORK AND INSTALLATION WHO IS THOROUGHLY FAMILIAR WITH THE TYPE OF MATERIALS BEING INSTALLED AND THE BEST METHODS FOR THEIR INSTALLATION, AND WHO SHALL DIRECT ALL WORK BEING PERFORMED UNDER THESE SPECIFICATIONS. THIS PERSON SHALL HAVE A MINIMUM OF FIVE (5) YEARS EXPERIENCE INSTALLING NATIVE PLANT MATERIALS FOR WETLAND MITIGATION OR RESTORATION PROJECTS, UNLESS OTHERWISE ALLOWED BY THE LANDSCAPE DESIGNER, WETLAND BIOLOGIST AND/OR THE CITY OF BELLEVUE.
4. SHORELINE RESTORATION AND BULKHEAD REMOVAL SHALL BE DONE PER FINAL BULKHEAD REMOVAL PLANS AND SPECIFICATIONS PROVIDED AT BUILDING PERMIT SUBMITTAL. CIVIL ENGINEER SHALL REVIEW AT KEY TIMES DURING BULKHEAD REMOVAL, WALL REPLACEMENT AND ROCK PLACEMENT.
5. EXISTING FAILING WALL TO BE REPLACED WITH NEW REINFORCED CONCRETE WALL OCCUPYING SAME DIMENSIONS AND FOOT PRINT. SURVEYOR TO SURVEY ELEVATIONS OF WALLS PRIOR TO DEMOLITION.
6. ALL PLANTING AREAS SHALL BE OVER-EXCAVATED 12" FOR PLACEMENT OF 12" OF IMPORTED 3-WAY TOPSOIL (DEJONG'S). RETAIN SOME OF THE GRAVELS FOR REPLACEMENT AS MULCH - SEE SPECIFICATION 10 BELOW. AOA TO APPROVE TOPSOIL PRIOR TO PLACEMENT.
7. ALL PLANTS SHOULD BE INSTALLED BETWEEN DECEMBER 1ST AND MARCH 15TH.
8. INTERMEDIATE INSPECTIONS. ALL PLANTS SHALL BE INSPECTED AND APPROVED BY THE LANDSCAPE DESIGNER AND/OR WETLAND BIOLOGIST PRIOR TO INSTALLATION. CONDITION OF ROOTS OF A RANDOM SAMPLE OF PLANTS WILL BE INSPECTED, AS WELL AS ALL ABOVEGROUND GROWTH ON ALL PLANTS. ROOTS OF ANY BARE ROOT PLANTS, IF PERMITTED FOR USE, WILL BE INSPECTED. PLANT MATERIAL MAY BE APPROVED AT THE SOURCE, AT THE DISCRETION OF THE LANDSCAPE DESIGNER AND THE WETLAND BIOLOGIST, BUT ALL MATERIAL MUST BE RE-INSPECTED AND APPROVED ON THE SITE PRIOR TO INSTALLATION. PLANT LOCATIONS SHALL ALSO BE INSPECTED AND APPROVED PRIOR TO PLANTING.
9. PRIOR TO INSTALLATION OF PLANT MATERIAL, THE PLANTING AREAS WILL BE LAID OUT BASED ON THE PLANTING PLAN, AND ALL HIMALAYAN BLACKBERRY, ENGLISH IVY OR OTHER INVASIVE PLANT SPECIES LOCATED IN THE PLANTING AREAS WILL BE REMOVED BY HAND.
10. ALL PLANTS SHALL BE PIT-PLANTED IN PLANTING PITS EXCAVATED 2X THE DIAMETER OF THE PLANT. PLANTS SHALL BE INSTALLED 3" HIGH AND SURFACED MULCHED TO A DEPTH OF 3" WITH BEACH GRAVELS REMOVED FROM THE PLANTING AREA WITHIN THE 100-YEAR FLOODPLAIN, OUTSIDE THE 100-YEAR FLOODPLAIN, MULCH WITH MEDIUM-COURSE BARK MULCH PLACED CONTINUOUSLY THROUGHOUT THE PLANTING BED.
11. ALL PLANTS SHALL BE NURSERY GROWN (IN WESTERN WA OR OR) FOR AT LEAST 1 YEAR FROM PURCHASE DATE, FREE FROM DISEASE OR PESTS, WELL-ROOTED, BUT NOT ROOT-BOUND AND TRUE TO SPECIES.
12. PLANT LAYOUT SHALL BE APPROVED BY AOA PRIOR TO INSTALLATION AND APPROVED UPON COMPLETION OF PLANTING.
13. UPON COMPLETION OF PLANTING, ALL PLANTS SHALL BE THOROUGHLY WATERED.
14. UPON APPROVAL OF PLANTING INSTALLATION BY AOA, THE CITY OF BELLEVUE WILL BE NOTIFIED TO CONDUCT A SITE REVIEW FOR FINAL APPROVAL OF CONSTRUCTION.
15. MAINTENANCE SHALL BE REQUIRED IN ACCORDANCE WITH THE CITY OF BELLEVUE SENSITIVE AREAS MITIGATION GUIDELINES AND APPROVED PLANS.
16. DESIGN-BUILD IRRIGATION SYSTEM BY LANDSCAPE CONTRACTOR TO PROVIDE FULL COVERAGE TO ALL PLANTING AREAS.
17. THE ZONE TO THE PLANTING BEDS SHALL BE SET TO PROVIDE 1/2" OF FLOW 2-3 TIMES WEEKLY FROM JUNE 15 - SEPTEMBER 30 THE FIRST YEAR AFTER PLANTING. FLOW SHALL REDUCE TO 1-2 TIMES WEEKLY THE SECOND YEAR AFTER PLANTING AND ONCE WEEKLY THE YEARS 3-5. NO FURTHER IRRIGATION IS NECESSARY AFTER THE THIRD YEAR FOR THE NATIVE PLANTING BEDS.
18. MAINTENANCE SHALL BE IMPLEMENTED ON A REGULAR BASIS ACCORDING TO THE SCHEDULE BELOW.

ANNUAL MAINTENANCE SCHEDULE

MAINTENANCE ITEM	J	F	M	A	M	J	J	A	S	O	N	D
WEED CONTROL			1		1	1	1	1	1	1		
GENERAL MAINT.			1		1	1	1	1	1	1		
WATERING - YEAR 1						4	8	8	8			
WATERING - YEAR 2						4	8	8	8			
WATERING - YEARS 3-5						4	4	4	4			

1-8 = NUMBER OF TIMES TASK SHALL BE PERFORMED PER MONTH.

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FIGURE 10: SPECIFICATIONS
BUFFER MITIGATION PLAN
672 W. LAKE SAMMAMISH PKWY. NE
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MAINTENANCE & MONITORING PLAN

CONSTRUCTION MANAGEMENT
1. Prior to commencement of any work in the steep slope and shoreline setback enhancement areas, the clearing limits will be staked and all existing vegetation to be saved will be clearly marked. A pre-installation meeting will be held at the site to review and discuss all aspects of the project with the owner.
2. A biologist will supervise plan implementation during construction to ensure that objectives and specifications of the steep slope and shoreline setback enhancement plan are met.
3. Any necessary significant modifications to the design that occur as a result of unforeseen site conditions will be jointly approved by the City of Bellevue and the biologist prior to their implementation.

MONITORING METHODOLOGY
1. The monitoring program will be conducted twice yearly (in the beginning and end of the growing season) for a period of five years, with reports submitted annually (at the end of the growing season) to the City of Bellevue.
2. Vegetation establishment within the steep slope and shoreline setback enhancement areas will be monitored during each field visit with a record kept of all plant species found.
3. Photo-points will be established from which photographs will be taken throughout the monitoring period. These photographs will document general appearance and progress in plant community establishment in the enhancement areas. Review of the photos over time will provide a semi-quantitative representation of success of the enhancement plan.

PERFORMANCE STANDARDS
Success of plant establishment within the steep slope and shoreline setback enhancement areas will be evaluated on the basis of percent survival of planted species.
1. Native woody cover will be a minimum of; 10% at construction completion, 15% at year 1, 20% at year 2, 25% at year 3 and 40% at year 5.
2. There will be 100% survival of all woody planted species throughout the mitigation planted area at the end of the first year of planting. For years 2-5, success will be based on an 85% survival rate or similar number of recolonized native woody plants.
3. Exotic and invasive plant species will be maintained at levels below 10% total cover. Removal of these species will occur immediately following the monitoring event in which they surpass the above maximum coverage. Removal will occur by hand whenever possible.

MAINTENANCE (M) & CONTINGENCY (C)
1. Established performance standards for the project will be compared to the monitoring results in order to judge the success of the enhancement project.
2. Contingency will include many of the items listed below and would be implemented if these performance standards are not met.
3. Maintenance and remedial action on the site will be implemented immediately upon completion of the monitoring event, (unless otherwise specifically indicated below).

- replace dead plants with the same species or a substitute species that meet the goal of the enhancement plan (C)
- re-plant areas after reason for failure has been identified (e.g., moisture regime, poor plant stock, disease, shade/sun conditions, wildlife damage, etc.) (C)
- irrigate following plant installation for five years (M)

PERFORMANCE BOND
1. A performance bond or other surety device will be posted with the City of Bellevue by the applicant to cover the costs of steep slope and shoreline setback enhancement plan implementation (including labor, materials, maintenance, and monitoring).
2. The bond or assignment may be released in partial amounts in proportion to work successfully completed over the five year monitoring period, as the applicant demonstrates performance and corrective measures.

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FIGURE 11: MAINTENANCE & MONITORING PLAN
BUFFER MITIGATION PLAN
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GEOTECH
CONSULTANTS, INC.

2401 10th Ave E
Seattle, Washington 98102
(425) 747-5618

June 8, 2017

JN 14468

Lago Mar, LLC
PO Box 7415
Bellevue, Washington 98008

Attention: Brian Heberling *via email: jawwws@outlook.com*

Subject: **Transmittal Letter – Geotechnical Engineering Study**
Proposed Residence
672 West Lake Sammamish Parkway Northeast
Bellevue, Washington

Dear Mr. Heberling:

We are pleased to present this geotechnical engineering report for the single-family residence to be constructed in Bellevue, Washington. The scope of our services consisted of exploring site surface and subsurface conditions, and then developing this report to provide recommendations for general earthwork and design criteria for foundations, retaining walls, and temporary shoring. This work was authorized by your acceptance of our proposal, P-9034, dated September 29, 2014. We had prepared another study for the project in late 2014, but the plans for the project have changed significantly; this report incorporates the new plans.

The attached report contains a discussion of the study and our recommendations. Please contact us if there are any questions regarding this report, or for further assistance during the design and construction phases of this project.

Respectfully submitted,

GEOTECH CONSULTANTS, INC.

D. Robert Ward, P.E.
Principal

DRW: mw

GEOTECHNICAL ENGINEERING STUDY
Proposed Single-Family Residence
672 West Lake Sammamish Parkway Northeast
Bellevue, Washington

This report presents the findings and recommendations of our geotechnical engineering study for the site of the proposed single-family residence to be located in Bellevue.

The site shoreline along Lake Sammamish has a northeast-southwest alignment. To simplify site descriptions, the project compass directions within this report refer to Lake Sammamish as due east of the site.

We have been provided with a topographic map that was updated in early 2016. We have also been provided with a site plan and a cross-section prepared by Alloy Design Group dated December 3, 2014. Based on this information plans, we understand that the existing house will be demolished and a four-story residence will be constructed with a similar footprint as the existing house. However, the new residence will extend further to the west. The residence will be set back no closer than about 8 feet from the north property line and about 12 feet from the south property line. It will be located about 8 to 10 feet east of the edge of an existing driveway. The basement level of the residence will be at approximately elevation 38 feet, which will require a cut of up to about 20 feet on the western side of the basement. The residence will have a garage at its third level, and a new small driveway will cross over an existing steep slope to connect the garage to the existing driveway that is west of the existing house. The driveway will be up to about 6 feet above the existing slope. In addition, a walkway is planned along about the eastern half of the northern side of the building; a retaining wall of up to about will be needed along the northern side of the walkway.

There is an existing rockery on the western edge of the existing driveway. It appears to have marginal stability, and we understand that the rockery will be removed and widened as part of this project. A new retaining wall is needed at the western side of the widened driveway.

If the scope of the project changes from what we have described above, we should be provided with revised plans in order to determine if modifications to the recommendations and conclusions of this report are warranted.

SITE CONDITIONS

SURFACE

The Vicinity Map, Plate 1, illustrates the general location of the site in East Bellevue. The large residential property extends east from a shared driveway off West Lake Sammamish Parkway down to Lake Sammamish. However, the western, approximate half of the property is mostly undeveloped and will not be altered for this project. The developed eastern portion of the property includes an existing house located near the lake and an existing driveway that extends in the north-south direction west of the house.

The existing house and associated landscaping are located east and downslope of the driveway. Directly east of the driveway is a steep approximately 16-foot-tall slope that includes a walkway, landscape walls, and landscaping. A relatively level bench is located at the toe of this slope, which is directly adjacent to the one-story house that is located within this bench. The house is located in a relatively level area and has a finish floor grade of approximately elevation 43 feet. East of the house the ground surface slopes somewhat moderately about 7 feet vertically down to Lake Sammamish. A few retaining walls with heights of up to 4 feet are located between the house and the lake and are composed of concrete and stacked concrete blocks. A dock extends from the site into the lake.

Several walls are located on the northern and western sides of the house area whose high sides are to the north and west. A timber wall with a height of about 9 feet is located within a foot of the north third of the west side of the house. This wall leans noticeably toward the house. In addition, a concrete retaining wall with a height of up to about 4 feet begins about 15 feet from the lake and continues toward the west, with a declining height in that direction. This wall appears to be on the adjacent property. A stacked block retaining wall rests on top of the concrete wall, and increases in height toward the west as the concrete wall becomes shorter. The block wall extends to about 60 feet from the lake. The maximum combined height of the two walls is about 6 feet.

There is a rockery and steep slope located adjacent to the west of the driveway. We did not observe indications of past instability of the steep slope. The rockery is approximately 6 to 8 feet tall, while the steep slope above is approximately 30 feet tall. The slope is mostly forested. The rockery is composed of rocks that are generally typical for one of its size. However, based on our experience, the rockery appears to be poorly constructed and is currently in a marginal stability condition. For example, it appears that some of the rocks have a depth of only about one foot, which is much smaller than normal, and they appear to have bulged outward. Although poorly built in our opinion, we did not observe, and are not aware of, any rocks that have fallen from the rockery.

The residences north and south of the house are set back about 10 feet from the adjoining property line and their lowest floors have elevations similar to the elevation of the existing site house. A landscape retaining wall that is about 3 to 5 feet tall is located on the northern property at the adjoining property line. The low side of the wall is at the grade of the subject property.

SUBSURFACE

The subsurface conditions were explored by drilling three test borings at the approximate locations shown on the Site Exploration Plan, Plate 2. Our exploration program was based on the proposed construction, anticipated subsurface conditions and those encountered during exploration, and the scope of work outlined in our proposal.

The borings were drilled on November 21, 2014 using a track-mounted, hollow-stem auger drill and a portable Acker drill that utilizes a small, gasoline-powered engine to advance a hollow-stem auger to the sampling depth. Samples were taken at approximate 2.5- and 5-foot intervals with a standard penetration sampler. This split-spoon sampler, which has a 2-inch outside diameter, is driven into the soil with a 140-pound hammer falling 30 inches. The number of blows required to advance the sampler a given distance is an indication of the soil density or consistency. A geotechnical engineer from our staff observed the drilling process, logged the test borings, and obtained representative samples of the soil encountered. The Test Boring Logs are attached as Plates 3 through 5.

Soil Conditions

Test Boring 1, located east and on the downslope side of the existing house, encountered approximately 8 feet of loose sand with gravel and silt. The sand then became less silty and medium-dense. Below a depth of 10.5 feet, medium-dense silt and silty sand was revealed that then became very dense at a depth of approximately 15 feet. The lowest sample in this test boring, taken at a depth of 20 feet, contained very dense sand.

Test Boring 2 was located within the existing driveway near the base of the rockery, while Test Boring 3 was located west of and about 30 feet vertically above the rockery. These test borings revealed medium-dense to dense, native sand with gravel and varying amounts of silt (mostly little) near the ground surface. This soil generally had a reduced gravel and silt in the 10-foot range and became dense to very dense below approximately 10 to 20 feet. This sand extended to the base of the boring at a depth of 26.5 feet.

Groundwater Conditions

Groundwater seepage was observed at a depth of approximately 13 to 18 feet in Test Borings 1 and 2 (lower test borings). No groundwater was encountered in Test Boring 3. The test borings were left open for only a short time period. Therefore, the seepage levels on the logs represent the location of transient water seepage and may not indicate the static groundwater level. Groundwater levels encountered during drilling can be deceptive, because seepage into the boring can be blocked or slowed by the auger itself.

It should be noted that groundwater levels vary seasonally with rainfall and other factors. At the eastern side of the site, we anticipate that groundwater could be found near the level of Lake Sammamish. In other areas to the west, it could be found near the observed grade noted in Test Boring 2 or perched on interbedded layers of silty soil.

The stratification lines on the logs represent the approximate boundaries between soil types at the exploration locations. The actual transition between soil types may be gradual, and subsurface conditions can vary between exploration locations. The logs provide specific subsurface information only at the locations tested. Where a transition in soil type occurred between samples in the borings, the depth of the transition was interpreted. The relative densities and moisture descriptions indicated on the test boring logs are interpretive descriptions based on the conditions observed during drilling.

CONCLUSIONS AND RECOMMENDATIONS

GENERAL

THIS SECTION CONTAINS A SUMMARY OF OUR STUDY AND FINDINGS FOR THE PURPOSES OF A GENERAL OVERVIEW ONLY. MORE SPECIFIC RECOMMENDATIONS AND CONCLUSIONS ARE CONTAINED IN THE REMAINDER OF THIS REPORT. ANY PARTY RELYING ON THIS REPORT SHOULD READ THE ENTIRE DOCUMENT.

The test boring located near the east edge of the proposed residence footprint encountered about 8 feet of loose soil which was underlain by competent soil consisting first of medium-dense silt and silty sand that became very dense at approximately 15 feet. The test boring in the driveway revealed medium-dense to dense sand at a shallow depth. Based on the test borings, it appears that a significant depth of loose soil underlies the eastern portion of the proposed residence. It is difficult to determine specifically with only these two borings, but we believe it is likely that the loose soil underlies approximately the eastern half of the basement level of the residence, while the competent soil underlies the western half. Because of this, we recommend that the eastern half be designed using a pile foundation, while the western half be designed for a conventional footing foundation (that bear on the competent soil). The actual interface of these two foundation types may be different, which can be specifically determined at the time of the initial project earthwork.

As noted earlier in this report, steep slopes are located above and below the existing driveway on the property. A discussion of the proposed development in relation to the steep slopes is discussed in a subsequent section of this report, and relevant sections of the City of Bellevue Code are addressed. In summary, however, we believe that this project is very suitable from a geotechnical engineering standpoint, including the locations of the new residence and the western retaining wall, provided the recommendations given in this report are followed.

Temporary excavations of 1:1 (Horizontal:Vertical) should be maintained for all excavations taller than 4 feet. If excavations cannot be kept within the property lines or cannot extend beyond the property line via an easement from an adjacent neighbor, temporary shoring will be needed. A discussion of cantilevered or tied-back soldier pile shoring is given in a subsequent section of this report. The height of the shoring wall on the western side of the will be such that cantilevered soldier piles may need to be spaced much closer than normal unless tied-back anchors are used. However, the constructability of the anchors could be difficult. The use of the anchors could be assessed if the soldier piles lengths do become too large. A retaining wall is needed near the northern property line in order to provide a walkway on the northern side of a portion of the residence. A permanent soldier-pile wall is likely the best type of retaining wall. The existing landscape wall that is just across the northern property line needs to be included in the design of this soldier-pile wall.

We understand the plan for the project is to remove the existing rockery and widen the western side of the driveway. Based on the very competent condition of the soil revealed in the nearby test borings, as well as the existence of a large, retaining/erosion control wall that is located on the northern neighboring property, it is our opinion that the soil at this location is quite stable. A new permanent wall is needed on the western edge of the driveway. The constructability of the new wall will be difficult. However, it appears as though this new retaining wall could likely be constructed using soil nails. The discussion of a soil nail wall on the western side of the driveway is included in a later section of this report.

The main level and garage level of the residence will be near the existing driveway grade. A new driveway is needed between the garage and the existing driveway. The driveway could be construction as a “bridge”, whereby there is vacant space under the driveway adjacent to the foundation of the residence, or fill can be placed under the driveway area. If fill is placed, it has to be accounted for in the pressure design of the western foundation wall of the residence. To lessen the impact of the needed fill, Geofoam could be used as fill. Geofoam is a Styrofoam material that provides excellent support but would only induce minimal pressure on the western foundation wall.

The erosion control measures needed during the site development will depend heavily on the weather conditions that are encountered. We anticipate that a silt fence will be needed around the downslope sides of any cleared areas. Existing pavements, ground cover, and landscaping should be left in place wherever possible to minimize the amount of exposed soil. Rocked staging areas and construction access roads should be provided to reduce the amount of soil or mud carried off the property by trucks and equipment. Trucks should not be allowed to drive off of the rock-covered areas. Cut slopes and soil stockpiles should be covered with plastic during wet weather. Following clearing or rough grading, it may be necessary to mulch or hydroseed bare areas that will not be immediately covered with landscaping or an impervious surface. On most construction projects, it is necessary to periodically maintain or modify temporary erosion control measures to address specific site and weather conditions.

The drainage and/or waterproofing recommendations presented in this report are intended only to prevent active seepage from flowing through concrete walls or slabs. Even in the absence of active seepage into and beneath structures, water vapor can migrate through walls, slabs, and floors from the surrounding soil, and can even be transmitted from slabs and foundation walls due to the concrete curing process. Water vapor also results from occupant uses, such as cooking and bathing. Excessive water vapor trapped within structures can result in a variety of undesirable conditions, including, but not limited to, moisture problems with flooring systems, excessively moist air within occupied areas, and the growth of molds, fungi, and other biological organisms that may be harmful to the health of the occupants. The designer or architect must consider the potential vapor sources and likely occupant uses, and provide sufficient ventilation, either passive or mechanical, to prevent a build up of excessive water vapor within the planned structure.

As with any project that involves demolition of existing site buildings and/or extensive excavation and shoring, there is a potential risk of movement on surrounding properties. This can potentially translate into noticeable damage of surrounding on-grade elements, such as foundations and slabs. However, the demolition, shoring, and/or excavation work could just translate into *perceived* damage on adjacent properties. Unfortunately, it is becoming more and more common for adjacent property owners to make unsubstantiated damage claims on new projects that occur close to their developed lots. Therefore, we recommend making an extensive photographic and visual survey of the project vicinity, prior to demolition activities, installing shoring, and/or commencing with the excavation. This documents the condition of buildings, pavements, and utilities in the immediate vicinity of the site in order to avoid, and protect the owner from, unsubstantiated damage claims by surrounding property owners. Additionally, any adjacent structures should be monitored during construction to detect soil movements. To monitor their performance, we recommend establishing a series of survey reference points to measure any horizontal deflections of the shoring system. Control points should be established at a distance well away from the walls and slopes, and deflections from the reference points should be measured throughout construction by survey methods.

Geotech Consultants, Inc. should be allowed to review the final development plans to verify that the recommendations presented in this report are adequately addressed in the design. Such a plan review would be additional work beyond the current scope of work for this study, and it may include revisions to our recommendations to accommodate site, development, and geotechnical constraints that become more evident during the review process.

We recommend including this report, in its entirety, in the project contract documents. This report should also be provided to any future property owners so they will be aware of our findings and recommendations.

STEEP SLOPE CONSIDERATIONS

The City of Bellevue Code defines Geologic Hazard Areas in section 20.25H.120. Specifically, a Steep Slope is defined as a slope of 40 percent or more that has a rise of at least 10 feet and exceeds 1,000 square feet in area. Under this definition, the slopes in the west and central portions of the site are classified as Steep Slopes.

The City defines a Landslide Hazard Area as any slope inclined at 15 percent or steeper that exhibits: a) areas of historic landsliding; b) soil movements within the Holocene Epoch (13,000 years ago to present); c) slopes that are parallel to sub-parallel to subsurface planes of weakness; d) slopes with geomorphic features indicating past historic movements; e) areas with spring seeps that indicate a shallow groundwater table on or adjacent to the slope face; and f) areas of potential instability from wave cutting, rapid stream incision, or stream bank erosion. We did not observe any signs of past or potential deep-seated landslide movements on this site, and we did not observe City-defined landslide criteria on the slope within this property. Medium-dense to dense soil was revealed in the test borings done on/near the site's steep slopes which became denser with depth. Thus, the core of the site is underlain be very stable, native, glacially-consolidated soil. Based on our observations of the slope, and the soil revealed in the test borings, the steep slopes within the site should not classify as a Landslide Hazard under this section of the City Code.

The Bellevue code, includes development restrictions for Geologic Hazard Areas, including a 75-foot structure setback drawn from the toe-of-slope (Section 20.25H.120.C.2b). However, Section Section 20.25H.120.C.3 indicates that the structure setback may be modified, and Section 20.25H.125 states that development within the steep slope and the structure setback may be allowed if all the provisions of this section are met, which identifies performance standards for development in setbacks. It is our professional opinion, because of the construction of new soil-nail wall at the western edge of the driveway where the toe-of-slope currently exists, which while greatly stabilize that steep slope, the structure setback can be reduced to 20 feet. The residence is proposed to be approximately 30 feet from the current toe-of-slope; therefore, provided the residence and soil-nail wall are constructed as recommended in this report, it is our opinion that construction of the project is very suitable from a geotechnical engineering perspective and meets the provisions of Section 20.25H.125. Thus, it is our opinion that the project can be constructed as it is currently planned with the new wall on the western edge of the existing driveway and a new residence to the east of the driveway.

SEISMIC CONSIDERATIONS

In accordance with the International Building Code (IBC), the site soil profile within 100 feet of the ground surface is best represented by Site Class Type D (Stiff Site Class). As noted in the USGS website, the mapped spectral acceleration value for a 0.2 second (S_s) and 1.0 second period (S_1) equals 1.28g and 0.49g, respectively.

The IBC states that a site-specific seismic study need not be performed provided that the peak ground acceleration be equal to $S_{DS}/2.5$, where S_{DS} is determined in ASCE 7. It is noted that S_{DS} is equal to $2/3S_{MS}$. S_{MS} equals F_a times S_S, where F_a is determined in Table 11.4-1. For our site, F_a = 1.0. The calculated peak ground acceleration that we utilized for the seismic-related parameters (earth pressures and seismic surcharges) of this report equals 0.34g. The core soils of the site (medium-dense/dense native soil) are not susceptible to seismic liquefaction because of their dense nature. Some loose soil revealed at the eastern edge of the site is susceptible to seismic liquefaction, but the residence foundation in that area will be supported on piles that will be embedded into competent soils below these potentially liquefiable soils.

EXCAVATIONS AND SLOPES

Excavation slopes should not exceed the limits specified in local, state, and national government safety regulations. Temporary cuts to a depth of about 4 feet may be attempted vertically in unsaturated soil, if there are no indications of slope instability. However, vertical cuts should not be made near property boundaries, or existing utilities and structures. Based upon Washington Administrative Code (WAC) 296, Part N, the soil at the subject site would generally be classified as Type B. Therefore, temporary cut slopes greater than 4 feet in height should not be excavated at an inclination steeper than 1:1 (Horizontal:Vertical), extending continuously between the top and the bottom of a cut.

The above-recommended temporary slope inclination is based on the conditions exposed in our explorations, and on what has been successful at other sites with similar soil conditions. It is possible that variations in soil and groundwater conditions will require modifications to the inclination at which temporary slopes can stand. Temporary cuts are those that will remain unsupported for a relatively short duration to allow for the construction of foundations, retaining walls, or utilities. Temporary cut slopes should be protected with plastic sheeting during wet weather. It is also important that surface runoff be directed away from the top of temporary slope cuts. Cut slopes should also be backfilled or retained as soon as possible to reduce the potential for instability. Please note that sand or loose soil can cave suddenly and without warning. Excavation, foundation, and utility contractors should be made especially aware of this potential danger. These recommendations may need to be modified if the area near the potential cuts has been disturbed in the past by utility installation, or if settlement-sensitive utilities are located nearby.

All permanent cuts into native soil should be inclined no steeper than 2:1 (H:V). Compacted fill slopes should also not be constructed with an inclination greater than 2:1 (H:V). To reduce the potential for shallow sloughing, fill must be compacted to the face of these slopes. This can be accomplished by overbuilding the compacted fill and then trimming it back to its final inclination. Adequate compaction of the slope face is important for long-term stability and is necessary to prevent excessive settlement of patios, slabs, foundations, or other improvements that may be placed near the edge of the slope.

Water should not be allowed to flow uncontrolled over the top of any temporary or permanent slope. All permanently exposed slopes should be seeded with an appropriate species of vegetation to reduce erosion and improve the stability of the surficial layer of soil.

Any disturbance to the existing slope outside of the building limits may reduce the stability of the slope. Damage to the existing vegetation and ground should be minimized, and any disturbed areas should be revegetated as soon as possible. Soil from the excavation should not be placed on the slope, and this may require the off-site disposal of any surplus soil.

SHORING FOR THE NEW RESIDENCE

As discussed above, the sensitivity of adjacent buildings and utilities must be considered in the design to reduce the risk of causing settlement of these adjacent elements. Regardless of the system used, all shoring systems will deflect in toward the excavation. Therefore, there is always a risk of noticeable settlement occurring on the ground behind the shoring wall. These risks are reduced, but not entirely eliminated, by using more rigid shoring systems, such as soldier piles.

The shoring design should be submitted to Geotech Consultants, Inc. for review prior to beginning site excavation. We are available and would be pleased to assist in this design effort. Cantilevered soldier pile shoring is generally suitable for wall heights up to the range of 15. However, as noted in the ***General*** section of this report, taller shoring walls can possibly be achieved by making the shoring spacing less than is typical or using tied-back anchors along with the soldier piles.

Soldier Pile Installation

Soldier pile walls would be constructed after making planned cut slopes, and prior to commencing the mass excavation, by setting steel H-beams in a drilled hole and grouting the space between the beam and the soil with concrete for the entire height of the drilled hole. There is water noted in the test borings and the soil is sandy, therefore the contractor should be prepared to case the holes or use the slurry method if caving soil is encountered. Excessive ground loss in the drilled holes must be avoided to reduce the potential for settlement on adjacent properties. If water is present in a hole at the time the soldier pile is poured, concrete must be tremied to the bottom of the hole.

As excavation proceeds downward, the space between the piles should be lagged with timber, and any voids behind the timbers should be filled with pea gravel, or a slurry comprised of sand and fly ash. Treated lagging is usually required for permanent walls, while untreated lagging can often be utilized for temporary shoring walls. Temporary vertical cuts will be necessary between the soldier piles for the lagging placement. The prompt and careful installation of lagging is important, particularly in loose or caving soil, to maintain the integrity of the excavation and provide safer working conditions. Additionally, care must be taken by the excavator to remove no more soil between the soldier piles than is necessary to install the lagging. Caving or overexcavation during lagging placement could result in loss of ground on neighboring properties. Timber lagging should be designed for an applied lateral pressure of 30 percent of the design wall pressure, if the pile spacing is less than three pile diameters. For larger pile spacings, the lagging should be designed for 50 percent of the design load.

Soldier Pile Wall Design

Soldier pile shoring has the same design whether there is one tie-back anchor or no anchor. The shoring that has a level backslope, such as on the northern and southern sides of the residence, should be designed for an active soil pressure equal to that pressure exerted by an equivalent fluid with a unit weight of 30 or 35 pounds per cubic foot (pcf) depending on whether the shoring is temporary or permanent, respectively. As noted earlier, the existing walls just across the northern property line need to be included in the design of any shoring wall that is near the northern property line. For the western shoring wall of the new residence, the pressures should be increased to 40 and 45 pcf, respectively, due to the adjacent above the western shoring walls. Traffic surcharges can typically be accounted for by increasing the effective height of the shoring wall by 2 feet.

It is important that the shoring design provides sufficient working room to drill and install the soldier piles, without needing to make unsafe, excessively steep temporary cuts. Cut slopes should be planned to intersect the backside of the drilled holes, not the back of the lagging.

Lateral movement of the soldier piles below the excavation level will be resisted by an ultimate passive soil pressure equal to that pressure exerted by a fluid with a density of 550 pcf. This soil pressure is valid only for a level excavation in front of the soldier pile; it acts on two times the grouted pile diameter. It is typical that a safety factor of 1.2 or 1.5 be included in the design of the shoring depending on whether it is temporary or permanent. A seismic surcharge analysis is also needed if the shoring is permanent, as noted in the **Foundation and Retaining Walls** section of this report..

Cut slopes made in front of shoring walls significantly decrease the passive resistance. This includes temporary cuts necessary to install internal braces or rakers. The minimum embedment below the floor of the excavation for cantilever soldier piles should be equal to the height of the "stick-up."

The vertical capacity of soldier piles to carry the downward component of the tieback forces will be developed by a combination of frictional shaft resistance along the embedded length and pile end-bearing.

PARAMETER	*DESIGN VALUE*
Pile Shaft Friction	1,500 psf
Pile End-Bearing	20,000 psf

Where: psf is Pounds per Square Foot.

The above values assume that the excavation is level in front of the soldier pile and that the bottom of the pile is embedded a minimum of 10 feet below the floor of the excavation. ((For the pile end-bearing to be appropriate, the bottom of the drilled holes must be cleaned of loosened soil.)) The shoring contractor should be made aware of this, as it may affect their installation procedures. The concrete surrounding the embedded portion of the pile must have sufficient bond and strength to transfer the vertical load from the steel section through the concrete into the soil.

TIEBACK ANCHORS

We recommend installing tieback anchors at inclinations between 20 and 30 degrees below horizontal. The tieback will derive its capacity from the soil-grout strength developed in the soil behind the no-load zone; this zone beings at the base of the excavation, extending horizontally 0.25H (H is the shoring height), and then up at an angle of 60 degrees from horizontal. The no-load zone is the area behind which the entire embedment length of each tieback anchor should be located. To prevent excessive loss-of-ground in a drilled hole, the no-load section of the drilled tieback hole should be backfilled with a sand and fly ash slurry, after protecting the anchor with a bond breaker, such as plastic casing, to prevent loads from being transferred to the soil in the no-load zone. The no-load section could be filled with grout after anchor testing is completed.

During the design process, the possible presence of foundations or utilities close to the shoring wall must be evaluated to determine if they will affect the configuration and length of the tiebacks.

Based on the results of our analyses and our experience at other construction sites, we suggest using an adhesion value of 2000 psf in the dense sand to design temporary anchors, if the mid-point of the grouted portion of the anchor is more than 10 feet below the overlying ground surface. This value applies to non-pressure-grouted anchors. Pressure-grouted or post-grouted anchors can often develop adhesion values that are two to three times higher than that for non-pressure-grouted anchors. These higher adhesion values must be verified by load testing.

Soil conditions, soil-grout adhesion strengths, and installation techniques typically vary over any site. This sometimes results in adhesion values that are lower than anticipated. Therefore, we recommend substantiating the anchor design values by load-testing all tieback anchors. At least two anchors in each soil type encountered should be performance-tested to 200 percent of the design anchor load to evaluate possible anchor creep. Wherever possible, the no-load section of these tiebacks should not be grouted until the performance tests are completed. Unfavorable results from these performance tests could require increasing the lengths of the tiebacks. The remaining anchors should be proof-tested to at least 135 percent of their design value before being "locked off." After testing, each anchor should be locked off at a prestress load of 80 to 100 percent of its design load.

If caving or water-bearing soil is encountered, the installation of tieback anchors will be hampered by caving and soil flowing into the holes. It will be necessary to case the holes, if such conditions are encountered. Alternatively, the use of a hollow-stem auger with grout pumped through the stem as the auger is withdrawn would be satisfactory, provided that the injection pressure and grout volumes pumped are carefully monitored.

All drilled installations should be grouted and backfilled immediately after drilling. No drilled holes should be left open overnight.

SOIL NAIL WALL FOR DRIVEWAY

Soil nailing is a shoring system where closely spaced, tieback anchors (nails) are grouted into drilled holes in the cut face as the excavation proceeds, thereby reinforcing the cut face. More anchors are required for this system than for conventional systems, but steel soldier piles and timber lagging are eliminated. The anchored or nailed system essentially operates as a reinforced soil wall or a gravity wall, with the nails tying the soil mass together.

The process begins by making a vertical cut in the shoring area followed immediately by the placement of anchors. Typical cuts for soil nails is generally 4 to 6 feet in height. Because the near-surface soils in area of this proposed soil nail wall is loose and the overall soils are quite sandy, we believe it is prudent to limit the first two lifts of excavation to approximately 4 feet. The cut face is then covered with a wire mesh, and shotcrete is placed over the mesh and soil face. Generally, no temporary, unsupported excavations for soil-nail walls should be allowed to stand longer than 12 hours without the acceptance of the geotechnical engineer. Once the shotcrete has hardened, the excavation again proceeds and the nails are placed. A geotextile drainage composite must be placed over the face of the cut prior to shotcreting to prevent buildup of hydrostatic pressures behind the shotcrete facing. As the excavation progresses downward, the drainage composite strips are extended, until reaching the base of the excavation, where weep holes are placed through the shotcrete to be tied into an underslab footing drain. After placing drainage composite and reinforcing, each lift face is then shot with a layer of concrete (shotcrete) before nail installation and excavation for the next lift begins.

Because soil nails are passive elements (they are not pre-stressed as tiebacks are, soil-nail walls will typically deflect more than a soldier-pile wall. This involves more risk of causing damage to adjoining utilities, streets, and other on-grade elements. The shoring designer should provide an estimate of the lateral deflection that is anticipated for the soil nail wall.

An important consideration of the soil nail wall construction is the potential for the temporary cut face to slough off after completing the cut. This is typically due to groundwater seepage exiting the face of the cut. It can also occur if larger pockets of fill soils are encountered, such as alongside utilities. This can increase the cost and time necessary to install the nailed wall. We recommend that the shoring contractor be consulted regarding potential difficulties and modifications that can occur during the construction of a soil-nailed wall.

Before commencing excavation, it is necessary to ensure that all surface water will be controlled during the excavation. Where face stability is problematic, a stabilizing berm can be left in place until the nail has been installed, or a ‘flash’ coat of shotcrete applied immediately following excavation. This may increase the cost and time necessary to install the soil nail wall.

The shoring designer will likely utilize one of several commercially available computer programs to design the nailed walls. We recommend that the following soil strength parameters be used in the nail wall design:

SOIL TYPE	MOIST UNIT WEIGHT (pcf)	EFFECTIVE INTERNAL FRICTION ANGLE (degrees)	EFFECTIVE COHESION (psf)	ALLOWABLE ADHESION (psf)
Near-surface loose soils (4 feet)	125	28	50	500
Lower sand soil	140	38	50	1,500

The adhesion values above should be substantiated by load-testing at least two soil nail per soil type to 200 percent of their design capacity, prior to installing production anchors. During shoring construction, at least 5 percent of the production anchors should be proof-tested to 130 percent of the design capacity.

EXCAVATION AND SHORING MONITORING

As with any shoring system, there is a potential risk of greater-than-anticipated movement of the shoring and the ground outside of the excavation. This can translate into noticeable damage of surrounding on-grade elements, such as foundations and slabs. Therefore, we recommend making an extensive photographic and visual survey of the project vicinity, prior to demolition activities, installing shoring or commencing excavation. This documents the condition of buildings, pavements, and utilities in the immediate vicinity of the site in order to avoid, and protect the owner from, unsubstantiated damage claims by surrounding property owners.

Additionally, the shoring walls should be monitored during construction to detect soil movements. To monitor their performance, we recommend establishing a series of survey reference points to measure any horizontal deflections of the shoring system. Control points should be established at a distance well away from the walls and slopes, and deflections from the reference points should be measured throughout construction by survey methods. At least two soldier piles should be monitored by taking readings at the top of the pile. Additionally, benchmarks installed on the surrounding buildings should be monitored for at least vertical movement. We suggest taking the readings at least once a week, until it is established that no deflections are occurring. The initial readings for this monitoring should be taken before starting any demolition or excavation on the site.

CONVENTIONAL FOUNDATIONS

As noted in the ***General*** section of this report, it appears that approximately the western half of the proposed residence can be supported on conventional continuous and spread footings bearing on undisturbed, medium-dense to dense (or denser), native soil. We recommend that continuous and individual spread footings have minimum widths of 12 and 16 inches, respectively. Exterior footings should also be bottomed at least 18 inches below the lowest adjacent finish ground surface for protection against frost and erosion. The local building codes should be reviewed to determine if different footing widths or embedment depths are required. Footing subgrades must be cleaned of loose or disturbed soil prior to pouring concrete. Depending upon site and equipment constraints, this may require removing the disturbed soil by hand.

An allowable bearing pressure of 3,000 pounds per square foot (psf) is appropriate for footings supported on competent native soil. A one-third increase in this design bearing pressure may be used when considering short-term wind or seismic loads. For the above design criteria, it is anticipated that the total post-construction settlement of footings founded on competent native soil will be about one-half inch, with differential settlements on the order of one-half inch in a distance of 30 feet along a continuous footing with a uniform load.

Lateral loads due to wind or seismic forces may be resisted by friction between the foundation and the bearing soil, or by passive earth pressure acting on the vertical, embedded portions of the foundation. For the latter condition, the foundation must be either poured directly against relatively level, undisturbed soil or be surrounded by level, well-compacted fill. We recommend using the following ultimate values for the foundation's resistance to lateral loading:

PARAMETER	ULTIMATE VALUE
Coefficient of Friction	0.45
Passive Earth Pressure	300 pcf

Where: pcf is Pounds per Cubic Foot, and Passive Earth Pressure is computed using the equivalent fluid density.

If the ground in front of a foundation is loose or sloping, the passive earth pressure given above will not be appropriate. We recommend maintaining a safety factor of at least 1.5 for the foundation's resistance to lateral loading, when using the above ultimate values.

PIPE PILES

We believe the driven pipe piles are very suitable for a deep foundation system needed on the eastern portion of the proposed residence. Three- or 4-inch-diameter pipe piles driven with a 650- or 800- or 1,100-pound hydraulic jackhammer to the following final penetration rates may be assigned the following compressive capacities.

INSIDE PILE DIAMETER	FINAL DRIVING RATE (650-POUND HAMMER)	FINAL DRIVING RATE (800-POUND HAMMER)	FINAL DRIVING RATE (1,100-POUND HAMMER)	ALLOWABLE COMPRESSIVE CAPACITY
3 inches	12 sec/inch	10 sec/inch	6 sec/inch	6 tons
4 inches	20 sec/inch	15 sec/inch	10 sec/inch	10 tons

> **Note:** The refusal criteria indicated in the above table are valid only for pipe piles that are installed using a hydraulic impact hammer carried on leads that allow the hammer to sit on the top of the pile during driving. If the piles are installed by alternative methods, such as a vibratory hammer or a hammer that is hard-mounted to the installation machine, numerous load tests to 200 percent of the design capacity would be necessary to substantiate the allowable pile load. The appropriate number of load tests would need to be determined at the time the contractor and installation method are chosen.

As a minimum, Schedule 40 pipe should be used. The site soils should not be highly corrosive. Considering this, it is our opinion that standard “black” pipe can be used, and corrosion protection, such as galvanizing, is not necessary for the pipe piles.

Pile caps and grade beams should be used to transmit loads to the piles. Isolated pile caps should include a minimum of two piles to reduce the potential for eccentric loads being applied to the piles. Subsequent sections of pipe can be connected with slip or threaded couplers, or they can be welded together. If slip couplers are used, they should fit snugly into the pipe sections. This may require that shims be used or that beads of welding flux be applied to the outside of the coupler.

Lateral loads due to wind or seismic forces may be resisted by passive earth pressure acting on the vertical, embedded portions of the foundation. For this condition, the foundation must be either poured directly against relatively level, undisturbed soil or be surrounded by level compacted fill. We recommend using a passive earth pressure of 300 pounds per cubic foot (pcf) for this resistance. If the ground in front of a foundation is loose or sloping, the passive earth pressure given above will not be appropriate. We recommend a safety factor of at least 1.5 for the foundation's resistance to lateral loading, when using the above ultimate passive value. Due to their small diameter, the lateral capacity of vertical pipe piles is relatively small. However, if lateral resistance in addition to passive soil resistance is required, we recommend driving battered piles in the same direction as the applied lateral load. The lateral capacity of a battered pile is equal to one-half of the lateral component of the allowable compressive load, with a maximum allowable lateral capacity of 1,000 pounds. The allowable vertical capacity of battered piles does not need to be reduced if the piles are battered steeper than 1:5 (Horizontal:Vertical).

FOUNDATION AND RETAINING WALLS

Retaining walls backfilled on only one side should be designed to resist the lateral earth pressures imposed by the soil they retain. The following recommended parameters are for walls that restrain level backfill:

PARAMETER	VALUE
Active Earth Pressure *	35 pcf
Passive Earth Pressure	300 pcf
Soil Unit Weight	130 pcf

Where: pcf is Pounds per Cubic Foot, and Active and Passive Earth Pressures are computed using the equivalent fluid pressures.

*** For a restrained wall that cannot deflect at least 0.002 times its height, a uniform lateral pressure equal to 10 psf times the height of the wall should be added to the above active equivalent fluid pressure.**

The design values given above do not include the effects of any hydrostatic pressures behind the walls and assume that no surcharges, such as those caused by slopes, vehicles, or adjacent foundations will be exerted on the walls. If these conditions exist, those pressures should be added to the above lateral soil pressures. Where sloping backfill is desired behind the walls, we will need to be given the wall dimensions and the slope of the backfill in order to provide the appropriate design earth pressures. The surcharge due to traffic loads behind a wall can typically be accounted for by adding a uniform pressure equal to 2 feet multiplied by the above active fluid density. Heavy construction equipment should not be operated behind retaining and foundation walls within a distance equal to the height of a wall, unless the walls are designed for the additional lateral pressures resulting from the equipment.

The values given above are to be used to design only permanent foundation and retaining walls that are to be backfilled, such as conventional walls constructed of reinforced concrete or masonry. It is not appropriate to use the above earth pressures and soil unit weight to back-calculate soil strength parameters for design of other types of retaining walls, such as soldier pile, reinforced earth, modular or soil nail walls. We can assist with design of these types of walls, if desired. The passive pressure given is appropriate only for a shear key poured directly against undisturbed native soil, or for the depth of level, well-compacted fill placed in front of a retaining or foundation wall. The values for friction and passive resistance are ultimate values and do not include a safety factor. We recommend a safety factor of at least 1.5 for overturning and sliding, when using the above values to design the walls. Restrained wall soil parameters should be utilized for a distance of 1.5 times the wall height from corners or bends in the walls. This is intended to reduce the amount of cracking that can occur where a wall is restrained by a corner.

Wall Pressures Due to Seismic Forces

The surcharge wall loads that could be imposed by the design earthquake can be modeled by adding a uniform lateral pressure to the above-recommended active pressure. The recommended surcharge pressure is 8**H** pounds per square foot (psf), where **H** is the design retention height of the wall. Using this increased pressure, the safety factor against sliding and overturning can be reduced to 1.2 for the seismic analysis.

Retaining Wall Backfill and Waterproofing

Backfill placed behind retaining or foundation walls should be coarse, free-draining structural fill containing no organics. This backfill should contain no more than 5 percent silt or clay particles and have no gravel greater than 4 inches in diameter. The percentage of particles passing the No. 4 sieve should be between 25 and 70 percent. If the native sand is used as backfill, a drainage composite similar to Miradrain 6000 should be placed against the backfilled retaining walls. The drainage composites should be hydraulically connected to the foundation drain system. The later section entitled ***Drainage Considerations*** should also be reviewed for recommendations related to subsurface drainage behind foundation and retaining walls.

The purpose of these backfill requirements is to ensure that the design criteria for a retaining wall are not exceeded because of a build-up of hydrostatic pressure behind the wall. Also, subsurface drainage systems are not intended to handle large volumes of water from surface runoff. The top 12 to 18 inches of the backfill should consist of a compacted, relatively impermeable soil or topsoil, or the surface should be paved. The ground surface must also slope away from backfilled walls to reduce the potential for surface water to percolate into the backfill. Water percolating through pervious surfaces (pavers, gravel, permeable pavement, etc.) must also be prevented from flowing toward walls or into the backfill zone. The compacted subgrade below pervious surfaces and any associated drainage layer should therefore be sloped away. Alternatively, a membrane and subsurface collection system could be provided below a pervious surface.

It is critical that the wall backfill be placed in lifts and be properly compacted, in order for the above-recommended design earth pressures to be appropriate. The wall design criteria assume that the backfill will be well-compacted in lifts no thicker than 12 inches. The compaction of backfill near the walls should be accomplished with hand-operated equipment to prevent the walls from being overloaded by the higher soil forces that occur during compaction. The section entitled ***General Earthwork and Structural Fill*** contains additional recommendations regarding the placement and compaction of structural fill behind retaining and foundation walls.

The above recommendations are not intended to waterproof below-grade walls, or to prevent the formation of mold, mildew or fungi in interior spaces. Over time, the performance of subsurface drainage systems can degrade, subsurface groundwater flow patterns can change, and utilities can break or develop leaks. Therefore, waterproofing should be provided where future seepage through the walls is not acceptable. This typically includes limiting cold-joints and wall penetrations, and using bentonite panels or membranes on the outside of the walls. There are a variety of different waterproofing materials and systems, which should be installed by an experienced contractor familiar with the anticipated construction and subsurface conditions. Applying a thin coat of asphalt emulsion to the outside face of a wall is not considered waterproofing, and will only help to reduce moisture generated from water vapor or capillary action from seeping through the concrete. As with any project, adequate ventilation of basement and crawl space areas is important to prevent a build up of water vapor that is commonly transmitted through concrete walls from the surrounding soil, even when seepage is not present. This is appropriate even when waterproofing is applied to the outside of foundation and retaining walls. We recommend that you contact an experienced envelope consultant if detailed recommendations or specifications related to waterproofing design, or minimizing the potential for infestations of mold and mildew are desired.

The ***General***, ***Slabs-On-Grade***, and ***Drainage Considerations*** sections should be reviewed for additional recommendations related to the control of groundwater and excess water vapor for the anticipated construction.

SLABS-ON-GRADE

The building floors can be constructed as slabs-on-grade atop competent native soil, or on structural fill. The subgrade soil must be in a firm, non-yielding condition at the time of slab construction or underslab fill placement. Any soft areas encountered should be excavated and replaced with select, imported structural fill.

Even where the exposed soils appear dry, water vapor will tend to naturally migrate upward through the soil to the new constructed space above it. This can affect moisture-sensitive flooring, cause imperfections or damage to the slab, or simply allow excessive water vapor into the space above the slab. All interior slabs-on-grade should be underlain by a capillary break drainage layer consisting of a minimum 4-inch thickness of clean gravel or crushed rock that has a fines content (percent passing the No. 200 sieve) of less than 3 percent and a sand content (percent passing the No. 4 sieve) of no more than 10 percent. Pea gravel or crushed rock are typically used for this layer.

As noted by the American Concrete Institute (ACI) in the *Guides for Concrete Floor and Slab Structures*, proper moisture protection is desirable immediately below any on-grade slab that will be covered by tile, wood, carpet, impermeable floor coverings, or any moisture-sensitive equipment or products. ACI also notes that vapor *retarders* such as 6-mil plastic sheeting have been used in the past, but are now recommending a minimum 10-mil thickness for better durability and long term performance. A vapor retarder is defined as a material with a permeance of less than 0.3 perms, as determined by ASTM E 96. It is possible that concrete admixtures may meet this specification, although the manufacturers of the admixtures should be consulted. Where vapor retarders are used under slabs, their edges should overlap by at least 6 inches and be sealed with adhesive tape. The sheeting should extend to the foundation walls for maximum vapor protection. If no potential for vapor passage through the slab is desired, a vapor *barrier* should be used. A vapor barrier, as defined by ACI, is a product with a water transmission rate of 0.01 perms when tested in accordance with ASTM E 96. Reinforced membranes having sealed overlaps can meet this requirement.

In the recent past, ACI (Section 4.1.5) recommended that a minimum of 4 inches of well-graded compactable granular material, such as a 5/8-inch-minus crushed rock pavement base, be placed over the vapor retarder or barrier for their protection, and as a "blotter" to aid in the curing of the concrete slab. Sand was not recommended by ACI for this purpose. However, the use of material over the vapor retarder is controversial as noted in current ACI literature because of the potential that the protection/blotter material can become wet between the time of its placement and the installation of the slab. If the material is wet prior to slab placement, which is always possible in the Puget Sound area, it could cause vapor transmission to occur up through the slab in the future, essentially destroying the purpose of the vapor barrier/retarder. Therefore, if there is a potential that the protection/blotter material will become wet before the slab is installed, ACI now recommends that no protection/blotter material be used. However, ACI then recommends that, because there is a potential for slab curl due to the loss of the blotter material, joint spacing in the slab be reduced, a low shrinkage concrete mixture be used, and "other measures" (steel reinforcing, etc.) be used. ASTM E-1643-98 "Standard Practice for Installation of Water Vapor

Retarders Used in Contact with Earth or Granular Fill Under Concrete Slabs" generally agrees with the recent ACI literature.

We recommend that the contractor, the project materials engineer, and the owner discuss these issues and review recent ACI literature and ASTM E-1643 for installation guidelines and guidance on the use of the protection/blotter material.

The ***General***, ***Permanent Foundation and Retaining Walls***, and ***Drainage Considerations*** sections should be reviewed for additional recommendations related to the control of groundwater and excess water vapor for the anticipated construction.

DRAINAGE CONSIDERATIONS

It is possible that, based on the final design of the residence, permanent foundation walls will be constructed against the shoring walls. Where this occurs, a plastic-backed drainage composite, such as Miradrain, Battledrain, or similar, should be placed against the entire surface of the shoring prior to pouring the foundation wall. Weep pipes located no more than 6 feet on-center should be connected to the drainage composite and poured into the foundation walls or the perimeter footing. A footing drain installed along the inside of the perimeter footing will be used to collect and carry the water discharged by the weep pipes to the storm system. Isolated zones of moisture or seepage can still reach the permanent wall where groundwater finds leaks or joints in the drainage composite. This is often an acceptable risk in unoccupied below-grade spaces, such as parking garages. However, formal waterproofing is typically necessary in areas where wet conditions at the face of the permanent wall will not be tolerable. If this is a concern, the permanent drainage and waterproofing system should be designed by a specialty consultant familiar with the expected subsurface conditions and proposed construction.

Footing drains placed inside the building or behind backfilled walls should consist of 4-inch, perforated PVC pipe surrounded by at least 6 inches of 1-inch-minus, washed rock wrapped in a non-woven, geotextile filter fabric (Mirafi 140N, Supac 4NP, or similar material). At its highest point, a perforated pipe invert should be at least 6 inches below the level of a crawl space or the bottom of a floor slab, and it should be sloped slightly for drainage. Plate 7 presents typical considerations for footing drains and Plate 8 presents a typical shoring detail. All roof and surface water drains must be kept separate from the foundation drain system.

If the structure includes an elevator, it may be necessary to provide special drainage or waterproofing measures for the elevator pit. If no seepage into the elevator pit is acceptable, it will be necessary to provide a footing drain and free-draining wall backfill, and the walls should be waterproofed. If the footing drain will be too low to connect to the storm drainage system, then it will likely be necessary to install a pumped sump to discharge the collected water. Alternatively, the elevator pit could be designed to be entirely waterproof; this would include designing the pit structure to resist hydrostatic uplift pressures.

As a minimum, a vapor retarder, as defined in the ***Slabs-On-Grade*** section, should be provided in any crawl space area to limit the transmission of water vapor from the underlying soils. Crawl space grades are sometimes left near the elevation of the bottom of the footings. As a result, an outlet drain is recommended for all crawl spaces to prevent an accumulation of any water that may bypass the footing drains. Providing even a few inches of free draining gravel underneath the vapor retarder limits the potential for seepage to build up on top of the vapor retarder.

Groundwater was observed during our field work. If seepage is encountered in an excavation, it should be drained from the site by directing it through drainage ditches, perforated pipe, or French drains, or by pumping it from sumps interconnected by shallow connector trenches at the bottom of the excavation.

The excavation and site should be graded so that surface water is directed off the site and away from the tops of slopes. Water should not be allowed to stand in any area where foundations, slabs, or pavements are to be constructed. Final site grading in areas adjacent to building should slope away at least 2 percent, except where the area is paved. Surface drains should be provided where necessary to prevent ponding of water behind foundation or retaining walls. A discussion of grading and drainage related to pervious surfaces near walls and structures is contained in the ***Foundation and Retaining Walls*** section.

GENERAL EARTHWORK AND STRUCTURAL FILL

All building and pavement areas should be stripped of surface vegetation, topsoil, organic soil, and other deleterious material. It is important that existing foundations be removed before site development. The stripped or removed materials should not be mixed with any materials to be used as structural fill, but they could be used in non-structural areas, such as landscape beds.

Structural fill is defined as any fill, including utility backfill, placed under, or close to, a building, behind permanent retaining or foundation walls, or in other areas where the underlying soil needs to support loads. All structural fill should be placed in horizontal lifts with a moisture content at, or near, the optimum moisture content. The optimum moisture content is that moisture content that results in the greatest compacted dry density. The moisture content of fill is very important and must be closely controlled during the filling and compaction process.

The allowable thickness of the fill lift will depend on the material type selected, the compaction equipment used, and the number of passes made to compact the lift. The loose lift thickness should not exceed 12 inches. We recommend testing the fill as it is placed. If the fill is not sufficiently compacted, it can be recompacted before another lift is placed. This eliminates the need to remove the fill to achieve the required compaction. The following table presents recommended relative compactions for structural fill:

LOCATION OF FILL PLACEMENT	MINIMUM RELATIVE COMPACTION
Beneath slabs or walkways	95%
Filled slopes and behind retaining walls	90%
Beneath pavements	95% for upper 12 inches of subgrade; 90% below that level

Where: Minimum Relative Compaction is the ratio, expressed in percentages, of the compacted dry density to the maximum dry density, as determined in accordance with ASTM Test Designation D 1557-91 (Modified Proctor).

Structural fill that will be placed in wet weather should consist of a coarse, granular soil with a silt or clay content of no more than 5 percent. The percentage of particles passing the No. 200 sieve should be measured from that portion of soil passing the three-quarter-inch sieve.

LIMITATIONS

The conclusions and recommendations contained in this report are based on site conditions as they existed at the time of our exploration and assume that the soil and groundwater conditions encountered in the test borings are representative of subsurface conditions on the site. If the subsurface conditions encountered during construction are significantly different from those observed in our explorations, we should be advised at once so that we can review these conditions and reconsider our recommendations where necessary. Unanticipated conditions are commonly encountered on construction sites and cannot be fully anticipated by merely taking samples in test borings. Subsurface conditions can also vary between exploration locations. Such unexpected conditions frequently require making additional expenditures to attain a properly constructed project. It is recommended that the owner consider providing a contingency fund to accommodate such potential extra costs and risks. This is a standard recommendation for all projects.

The recommendations presented in this report are directed toward the protection of only the proposed structure from damage due to slope movement. Predicting the future behavior of steep slopes and the potential effects of development on their stability is an inexact and imperfect science that is currently based mostly on the past behavior of slopes with similar characteristics. Landslides and soil movement can occur on steep slopes before, during, or after the development of property. At additional cost, we can provide recommendations for reducing the risk of future movement on the steep slopes, which could involve regrading the slopes or installing subsurface drains or costly retaining structures. The owner of any property containing, or located close to steep slopes must ultimately accept the possibility that some slope movement could occur, resulting in possible loss of ground or damage to the facilities around the proposed building residence.

This report has been prepared for the exclusive use of Lago Mar, LLC and its representatives for specific application to this project and site. Our conclusions and recommendations are professional opinions derived in accordance with our understanding of current local standards of practice, and within the scope of our services. No warranty is expressed or implied. The scope of our services does not include services related to construction safety precautions, and our recommendations are not intended to direct the contractor's methods, techniques, sequences, or procedures, except as specifically described in our report for consideration in design. Our services also do not include assessing or minimizing the potential for biological hazards, such as mold, bacteria, mildew and fungi in either the existing or proposed site development.

ADDITIONAL SERVICES

Geotech Consultants, Inc. should be retained to provide geotechnical consultation, testing, and observation services during construction. This is to confirm that subsurface conditions are consistent with those indicated by our exploration, to evaluate whether earthwork and foundation construction activities comply with the general intent of the recommendations presented in this report, and to provide suggestions for design changes in the event subsurface conditions differ from those anticipated prior to the start of construction. However, our work would not include the supervision or direction of the actual work of the contractor and its employees or agents. Also, job and site safety, and dimensional measurements, will be the responsibility of the contractor.

During the construction phase, we will provide geotechnical observation and testing services when requested by you or your representatives. Please be aware that we can only document site work we actually observe. It is still the responsibility of your contractor or on-site construction team to verify that our recommendations are being followed, whether we are present at the site or not.

The following plates are attached to complete this report:

Plate 1	Vicinity Map
Plate 2	Site Exploration Plan
Plates 3 - 5	Test Boring Logs
Plate 6	Typical Footing Drain Detail
Plate 7	Typical Shoring Drain Detail

We appreciate the opportunity to be of service on this project. Please contact us if you have any questions, or if we can be of further assistance.

Respectfully submitted,

GEOTECH CONSULTANTS, INC.

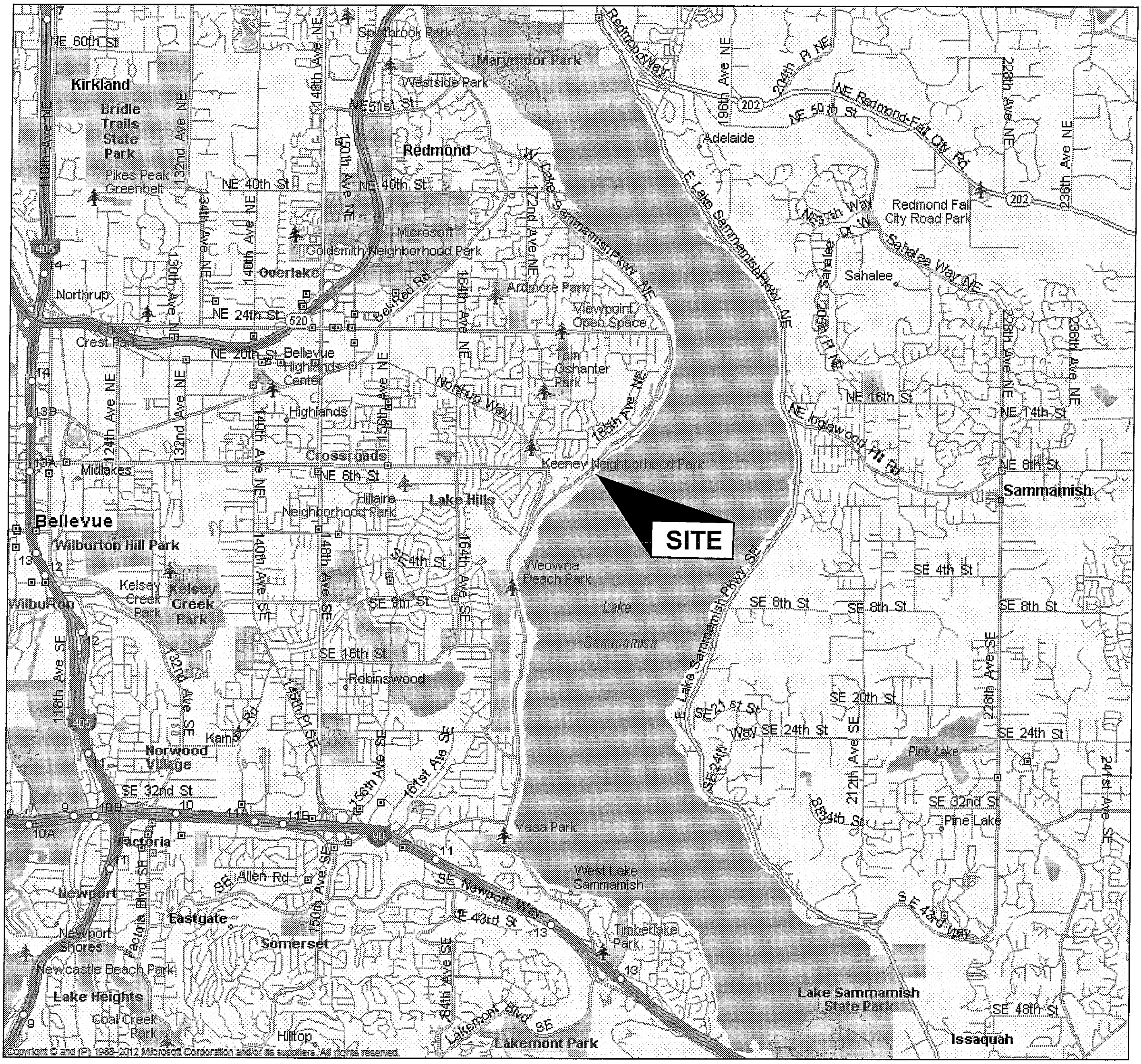
D. ROBERT WARD
STATE OF WASHINGTON
19740
REGISTERED
PROFESSIONAL ENGINEER

6/8/17

D. Robert Ward, P.E.
Principal

TRC/DRW:mw

NORTH



(Source: Microsoft MapPoint, 2013)

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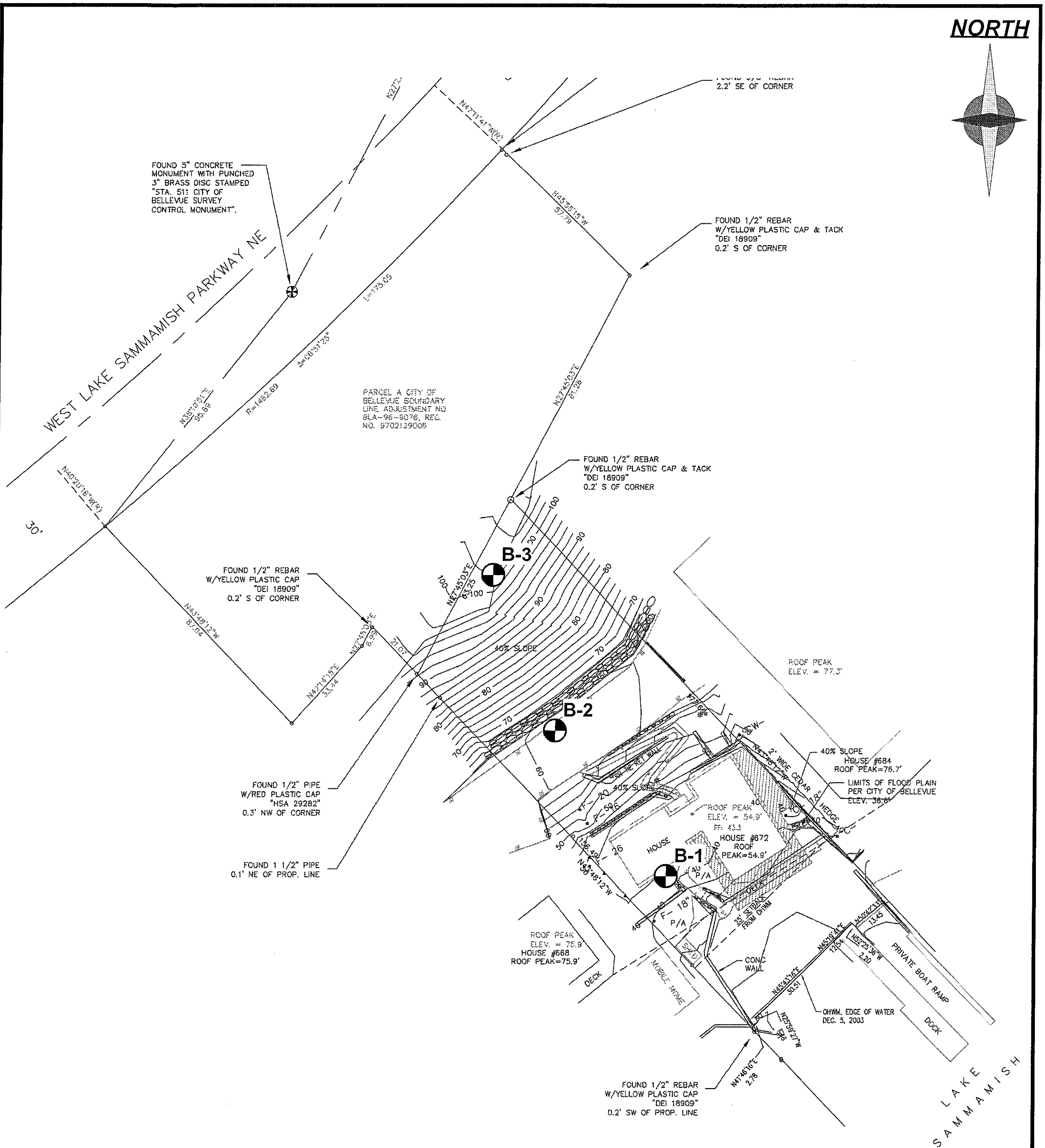
VICINITY MAP

672 West Lake Sammamish Parkway NE
Bellevue, Washington

Job No:	Date:		Plate:
14468	Dec. 2014		1



Legend:

Test Boring Location

GEOTECH CONSULTANTS, INC.

SITE EXPLORATION PLAN

672 West Lake Sammamish Parkway NE
Bellevue, Washington

Job No:	Date:		Plate:
14468	Dec. 2014	No Scale	2

BORING 1

Depth (ft.)	Moisture	Water Table	Blows per Foot	Sample	USCS	Description
5			6	1	SW	Brown SAND with gravel and silt, fine to coarse grained, moist, loose -becomes gray-brown and wet
10			21	2		-becomes medium-dense, reduced silt content
					ML	Gray SILT, non-plastic, moist, medium-dense
15			82/ 10.5 "	3	SM	Gray silty SAND, fine to medium-grained, moist, very dense
					ML	Gray SILT, non-plastic, moist, very dense
					PT	Dark brown PEAT, moist, very dense
		▼			ML	Gray SILT, non-plastic, moist, very dense
20			50/5"	4	SW	Gray SAND, fine to coarse-grained, wet, very dense
25						
30						

* **Test boring was terminated at 21 feet on November 21, 2014.**
* **Groundwater was encountered at 18 feet during drilling.**

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TEST BORING LOG

672 West Lake Sammamish Parkway NE
Bellevue, Washington

Job	Date:	Logged by:	Plate:
14468	Dec. 2014	TRC	3

BORING 2

Depth (ft.)	Moisture	Water Table	Blows per Foot	Sample	USCS	Description
0						4" asphalt
			27	1		Brown SAND with gravel and silt, fine to coarse-grained, moist, medium-dense
5			20	2		
10			33	3		-becomes dense, reduced gravel and silt content
		▼			SW	
15			36	4		-becomes wet
20			43	5		
25			48	6		-becomes gray, fine to medium-grained
30						

* **Test boring was terminated at 26.5 feet on November 21, 2014.**
* **Groundwater was encountered at approximately 13 feet during drilling.**

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TEST BORING LOG

672 West Lake Sammamish Parkway NE
Bellevue, Washington

Job	Date:	Logged by:	Plate:
14468	Dec. 2014	TRC	4

BORING 3

Depth (ft.)	Moisture	Water Table	Blows per Foot	Sample	USCS	Description
5			30	1	SM	Gray-brown silty SAND with gravel, fine to coarse-grained, moist, dense
10			21	2	SW	Brown SAND with gravel, fine to coarse-grained, moist, medium-dense
15			26	3		-reduced gravel content
20			60	4	ML	Rust-brown mottled gray-brown SILT, non-plastic, moist, very dense
					SW	Gray-brown SAND, fine to coarse-grained, moist, very dense
25			60	5		-increased gravel content
30						

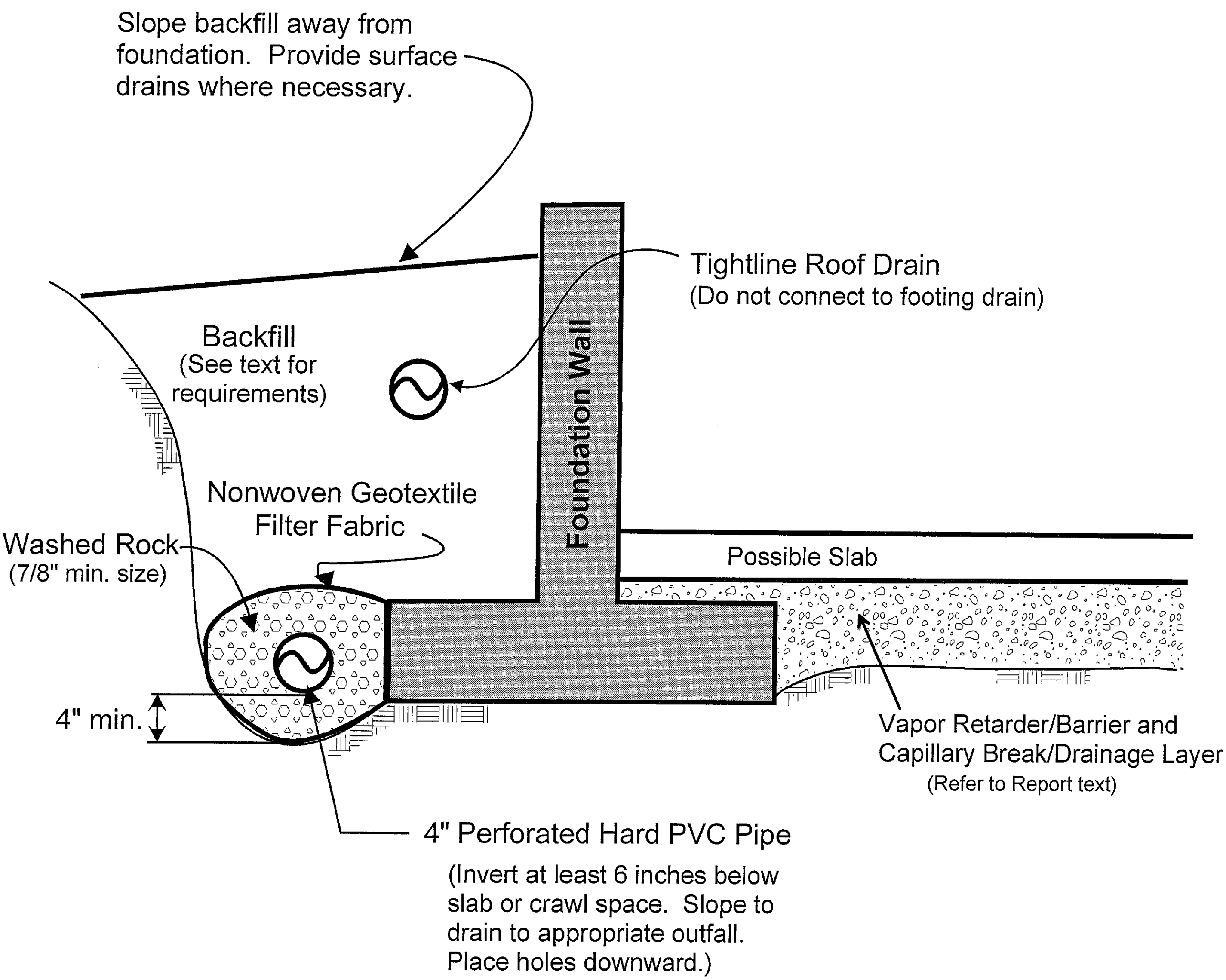
* **Test boring was terminated at 26.5 feet on November 21, 2014.**
* **No groundwater was encountered during drilling.**

GEOTECH CONSULTANTS, INC.

TEST BORING LOG

672 West Lake Sammamish Parkway NE
Bellevue, Washington

Job	*Date:*	*Logged by:*	*Plate:*
14468	Dec. 2014	TRC	5



NOTES:

(1) In crawl spaces, provide an outlet drain to prevent buildup of water that bypasses the perimeter footing drains.

(2) Refer to report text for additional drainage, waterproofing, and slab considerations.

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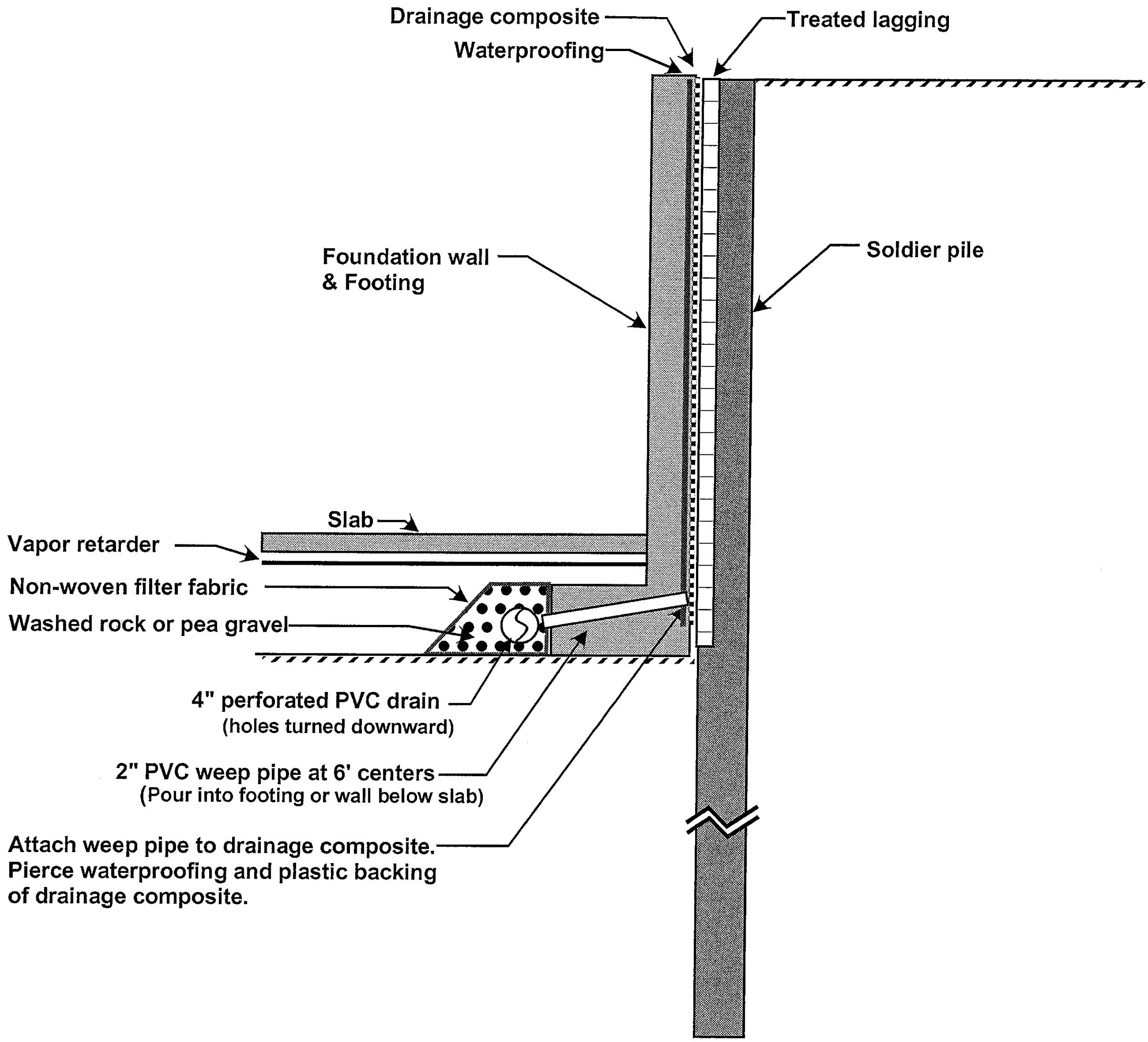
FOOTING DRAIN DETAIL

672 West Lake Sammamish Parkway NE
Bellevue, Washington

Job No:	Date:		Plate:
14468	Dec. 2014		6



Note - Refer to the report for additional considerations related to drainage and waterproofing.

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SHORING DRAIN DETAIL

672 West Lake Sammamish Parkway NE
Bellevue, Washington

Job No:	Date:		Plate:
14468	Dec. 2014		7